DESIGN OF A STEEL RAILROAD BRIDGE

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STEEL RAILROAD BRIDGE

A THESIS PRESENTED

BY

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D E S I G N O F A S T E E L R A I L R O A D B R I D G E

This structure consists of a single track railroad bridge, and is designed to span a river, having a width of 600 feet.

The design of this bridge, and the estimate of its cost may be divided into five parts:

1. Selection of design.
2. Determination of stresses.
3. Calculations of sections and weights.
4. Design of piers and abutments.
5. Cost.

1. SELECTION OF DESIGN

In designing this bridge, one of the first considerations was regarding the number of spans. This question was decided by the principle that the total cost of the substructure and superstructure shall be a minimum. In any event there will be two land abutments; and the relative cost of piers and their connecting spans determines the number of piers and spans which can be most economically built between two abutments.

The cost of bridges is closely proportional to their weights. If l be the length of one span, the formula $W= al+bl^2$ gives a good approximation to the weight, a and b being constants for the same type of truss. In this, al represents the weight of the track and floor system, while bl^2 represents the weight of the main trusses and lateral bracing.

If the cost of piers is about equal, and they be spaced at equal distance apart, the following investigation will give

the economical number of spans. Let L be thetotal distance between end abutments, x the number of spans, andhence x-1 the number of piers, m the cost of the two abutments, n the cost of each pier, p the cost per pound of the bridge superstructure. The weight of x spans, each of length L/x, is $x(aL/x+bL^2/x^2)$, and the total cost of the work is $C= m+n(x-1)+p(aL+bL^2/x)$ where C represents the total cost of the structure.

This will be a minimum when the first derivative of C with respect to x becomes zero, and this gives $N= pbL/x$ which shows that the cost of one of the intermediate piers should aqual the cost of the main and lateral trusses of one of the spans.

Or $x= pbL/n$ gives the economical number of spans.

In order to solve for x, it was necessary to make an approximate estimate of the cost of one pier and conditions governing it were as follows.

The height of the high and low water of the river was taken as 100 and 85 feet above a known bench of elevation. The base of rail was supposed to be located at a distance of 8 feet above the high water. The bed of the river consists of fine sand varying from 5-6 feet in thickness and underlaid by fine gravel which in its term is supported by layers of solid rock, occuring at an elevation of 70 feet above the referred bench.

In order to avoid driving of piles and, in the meantime, gives to the piers a solid foundation, the soil beneath the water will be excavated and the piers built on solid rock. The height of the pier was assumed to be 32 feet. The dimensions

of the pier consists of 9'x25' at the bottom and 6'x22' at the top; and the test of stability against sliding and overturning is determined in the following manner.

Stability against sliding.

Let the coping of the pier be assumed to be 7'x23' and its thickness one foot. The volume and weight of the pier will be found as follows:

Area of coping= 7x20+0.78x7x3= 156.38 square feet.

Volume of " = 156.38x1= 156.38 cubic feet.

Area of the top cross-section= 6x19+0.78x6x3= 127 square feet.

Area of the bottom cross-section= 9x22+0.78x9x3= 219.6 " "

Volume between the bottom and coping of the pier-

$\left(\frac{127+219.6}{2}\right)31$= 5545 cubic feet.

Total volume of the pier is therefore 5545+156.38= 5701.38 cubic feet.

The material to be used for construction of piers will consist of concrete, and assuming the weight of a cubic foot of concrete to be 150 pounds, the weight of the pier is therefore 150x5701.38= 855200#= 427.1 tons.

The length of one of the simple spans of the bridge was assumed to be 150 feet.

The pressure of the wind against the truss and train together was taken at 30 lbs. per square foot of truss and train.

The pressure of the wind against the truss alone was taken at 50 lbs. per square foot against twice thevertical projection of one truss, which for well proportioned trusses will average

about 10 square feet per linear foot of span. The exposed surface of a train was taken as 10 square feet per linear foot. The velocity of the stream was taken as 10 feet per second.

In consideration of the above mentioned items the wind pressure was computed in the following manner.

The wind surface= 10x150= 1500 square feet, and thewind pressure against the truss is 30x1500= 45000#= 22.5 tons.

The exposed surface of the pier from the low water to the top of pier is found to be $\left(\frac{24+22}{2}\right)26 = 598$ square feet.

The wind pressure against the pier is therefore 20x598-11960#= 5.9 tons.

The crushing strength of ice was assumed to be 15 tons per square foot, and the thickness of ice one foot. The pier is about 7 feet wide at the high water, the exposed area therefore equals, 7x1= 7 square feet, and the pressure produced on that area was considered to be 15x7= 105 tons.

The formula used in determining the pressure due to the action of current was $P = swkv^2/2g$

(See Baker's Masonry Construction p.367) where s= exposed surface, K= a coefficient taken as 1.1, w= weight of a cubic foot of water and v= the velocity of the stream.

In this design "s" was taken as 15x25= 375 square feet; "w" was assumed to be 62.5 lbs. and "v" as 10 ft/sec.

By substitution of these values the formula becomes

$$P = \frac{375 \times 62.5 \times 1.1 \times 100}{2 \times 64} = 20100\# = 10.5 \text{ tons.}$$

The summation of the above mentioned items is found as follows:

Pressure of wind on the truss.....................22.5 tons

" " " " " train....................22.5 "

" " " " " pier.................... 5.9 "

" " ice " " " ....105.0 "

" " current............................ 10.5 "

Total force tending to slide-..................166.4 "

The tendency of this force must be resisted by the weight of the trusses, that of empty cars, and also by the weight of masonry.

The weight per linear foot of bridge was calculated from the formula W= 650+71, where "l" is the length of the span.

The total weight of 150 foot span is therefore 650x150+ 7x150x150= 255000#= 127.5 tons.

The weight of empty cars was assumed to be one half of a ton per linear foot of span, and the total weight of cars is 0.5x150= 75 tons.

The total weight to resist sliding is therefore found to be 127.5+75+427.1=629.6 tons.

Sliding cannot take place, if the coefficient of friction is equal or greater than $\frac{166.4}{629.6}$= 0.264 which is within safe limits in this design.

Test for stability against overturning.

The forces that tend to produce sliding also tend to cause overturning, the paints of application of these forces were determined in the following manner.

The center ofpressure of the wind on the truss was assumed to be applied at the middle of its height; that of the wind on the train was taken as 8 feet above the top of the rail; and that of the wind on the pier at middle of the exposed part. The arm for the pressure of the ice was measured from the high water. The center of pressure of the current was assumed to be at one third of the depth. All the downward forces were considered to have been acted vertically through the center of the pier.

In consideration of the above mentioned data the overturning and resisting mements were computed as follows.

The pressure of the wind on the truss is 22.5 tons, and its lever arm- height of pier plus half the depth of the truss, which in this design was taken as 30 feet.

Therefore the moment of this force is 22.5x(32+15)= 1057.5 foot-tons.

The pressure of the wind on thetrain is also 22.5 tons and its lever arm= distance from the footing to the top of the pier plus the distance from the top of pier to the top of the rail plus the distance from the top of the latter to the center of train, and this equals 32+8+8= 48 feet.

The moment of this pressure is therefore 22.5x48= 1080 foot-tons.

The pressure of the wind against the pier was found to be 5.9 tons; the arm of this force was assumed to be applied at the center of the exposed surface and equals 23 feet.

Therefore the moment is 5.9x23= 135.7 foot-tons.

The pressure of the ice was found to be 105 tons, and its

arm was taken as 15 feet, which in this case is the distance from the high water.

The moment of this pressure 105x15= 1575 foot-tons.

The pressure of the water is 105 tons, and the depth of the water was assumed as 21 foot, therefore the arm of this force is 21/3= 7 feet.

The moment of this pressure is 10.5*7= 73.5 foot-tons.

The summation of all these moments is therefore

Moment of the wind on the truss...............	1057.5	foot-tons
" " " " " " train...............	1030.0	" "
" " " " " " pier................	135.7	" "
" " " pressure of the ice.............	1575.0	" "
" " " " " " current...........	73.5	" "
Total overturning moment.......................	3921.7	" "

This overturning moment must be resisted by the moment produced by the weight of the pier, trusses and that of empty cars. By taking moments at the toe whose distance is 12.5 feet from the center of the pier, the resisting moment is therefore equal 629.6x12.5= 7870 foot-tons.

A factor of safety is therefore found to be which equals 7870:3921.7= 2.06, that shows that the piers is safe enough for overturning.

After having found the right dimensions of the pier, it is also necessary to make an estimate of the cost of a cofferdam.

The length of the cofferdam was assumed to be 45 feet, the width of it was taken as 30 feet, while its height must be

sufficient to prevent the high water from flowing into the da m. In this case it is safe enough to assume theheight of the cofferdam to be 32 feet.

A cross-section of the later is shown on the next page, Fif. 1.

The description of the construction of the cofferdam may be outlined in the following manner.

The area to be inclosed is first surrounded by two rows of ordinary piles. On the outside of the main piles, a little below the top, are bolted two longitudinal pieces, and on the inside are fastened two similar pieces, which serve as guides for the sheet piles, while being driven.

A rod connects the tops of the opposite main piles to prevent spreading when the puddle is put in. Timber is put on primarily to carry the footway and so fastened as to prevent the puddle space from spreading.

In this design the main piles were assumed to be spaced 5 feet apart, which will require about 36 piles longitudinally and 24 ones transversely to the cofferdam. The cost of one pile including driving was considered to be \$3.20

The sheeting to be used was assumed to be 4"x8", and their cost for 1000 units of B.M. was taken as \$25.00

The volume of sheeting required is $1/3 \times 2(90x32)+2(54x32) =$ 3072 cubic feet.

The volume of soil to be excavated was found to be

$\frac{45x29x15}{27} =$ 726 cubic yards.

The cost of excavation of a cubic yard of soil was taken as \$1.25.

Section of Cofferdam.

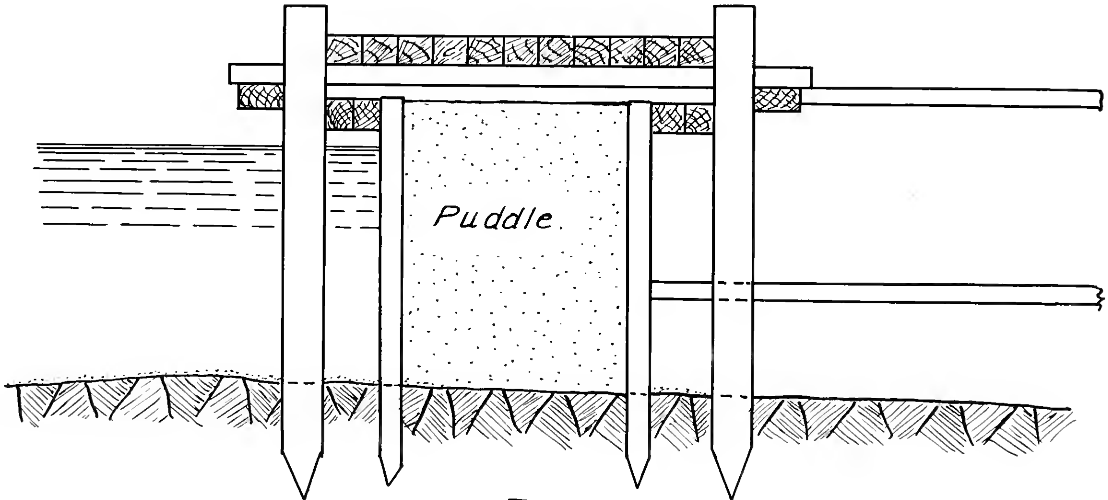


Fig 1.

The cost of laying sheeting was assumed as $5.00 per cubic yard, while the cost of one cubic yard of concrete was taken in this design as $5.50 and its laying was considered to cost about $1.30 per cubic yard of material.

The summation of the cost of the above mentioned different items will give the total cost of a pier, which is found from the following table.

Item	Amount	Price	Total
For concrete	209.8 cub.yd.	5.50	$1155.00
" laying concrete	209.8 " "	1.30	273.00
" piles	60 pieces	3.20	192.00
" sheeting	36.9 cub.yd.	25.00	922.50
" laying "	36.9 " "	5.00	194.50
" excavation	726 cub. yds.	1.25	907.50
Total cost...................................			$3644.50

The number of spans can be found next from the formula

$x = pbL/n$

The value of p in this design was taken as 4 cents per pound of metal. The total width of the river is 600 feet; and the value of 1 for pin-connected bridges is usually taken as .7. Therefore the number of spans required equals

$x = 0.04x7x600x600/3644.5 = 4$ or 5.

From the last expression it is seen that either four or five spans ought to give the most economical arrangement of spans. The cost of the structure under these conditions will be regarder and a comparison made in order to select the best design.

Let the cost of five spans and their piers be considered first. Since the total length of the bridge is 600 feet, a length of 120 feet will be assumed for each span. The weigh of one span was found from the formula $W= 650l+7l^2$ where l is the length of the span, in this case it is taken as 120 feet.

The total weight of five spans is 5/650x120+7x120x120= 894000 lbs.

If the cost of one pound of metal be taken as 4 cents, the cost of five spans will be .04x894000= $35760.

When five spans will be selected, there will be only four abutments, and their cost is 4x3644.5= $14578.

If the selection of the design be considered to be four spans, the length of each span is therefore 150 ft., and the total weight of these four spans will be 4(650x150+7x150x150= 1020000 lbs and their cost is therefore .04x1020000= $40800.

The cost of three piers will be 3x3644.5= $10933.5.

The total cost of the structure under the second case is 40800+10933.5= $51733.5.

Since the difference between the two cases isnot considerable, a selection of four spans will be of better practice and therefore it will be adopted.

The selection of the most economical arrangement of panels and depth of thetruss will be considered next.

Let the weight per foot per truss of the stringers, cross girders and depth of the track be represented by w_2, and the weight per foot per truss of the uniformily distributed load, equivalent to the live load assumed, w_1. Let the weight per foot of one main truss be w_4, and let w_0 be the weight per

per foot of lattice, bars, pins, eye bar heads, cover plates, rivets etc. Then the total load per foot per truss, is $w_1 + w_0 + w_3 + w_4$. Let the length of the panel be p then $(w_1 + w_0 + w_2 + w_3 + w_4)p$ will be the total panel load for one truss. Let N be the number of panels, d= the depth in feet, and l= the span in feet.

The formula for the weight per foot of one main truss is given on p. 487, DuBois Mechanics of Engineering V. 2, which is as follows:

$$w_4 = \frac{w_1 + w_0 + w_2 + w_3}{\dfrac{3.6\, f d}{\alpha p^2 + \beta d^2 + \dfrac{\alpha p^2 + 4\beta d^2}{5(N-1)}} - 1} \qquad (1)$$

where α, β are constants depending on the form of truss, and f is the numeration of Gordon's formula.

The values of w_0 w_2 and w_3 are rational ones and are also given in terms of known quantities on p. 484-492 of the book of the above mentioned author. They are as follows.

$$w_3 = \frac{3Nl + 568}{170}\Big/p$$

$w_0 = \dfrac{Nd}{3} + 0.875N(12-N) + 6$, while the values of w_2 for different number of panels are given on p. 490 of the same book.

If the formula (1) is reliable, and gives even with tolerable accuracy the weight of truss then, since it is rational in form, the depth, which gives the least weight can also be determined.

Differentiating and putting the first differential equal to zero, the formula is therefore.

$$\frac{d}{l} = \frac{1}{N}\sqrt{\frac{\alpha\left(1 + \dfrac{1}{5(N-1)}\right)}{\beta\left[1 + \dfrac{4}{5(N-1)}\right] + \dfrac{1.2 f N}{(w_1 + w_2 + w_3 + w_4) + A}}} \qquad (2)$$

Calling the right part of this equation C, the formula becomes d= Cl.

The values for the above mentioned constants were taken from Du Bois Mechanics of Engineering and arranged in talular form as follows:

N	W_0	W_2	W_3	W_4	C
5	93	395	32.1	198	0.2258
6	98.04	376	38	201	0.2018
7	101.4	361	37	221	0.1846

For constructive reasons it is best to limit the length of panel to about 30 feet and the depth of truss to 50 feet. Within these limits it is possible to find thebest number of panels and depth by trail. The sum of $w_1 + w_2 + w_3 + w_0$ can be taken as 2000 pounds without no ticeable error, and the total weight per foot of all the metal is $2(w_2 + w_3 + w_4 + w_0 - 200)$.

The value of N which gives a minimum is thebest.

If the number of panels be 5, the panel length will therefore be 150/5=30 feet, and the depth is Cl= 0.2018x 150= 30.2 feet.

The total weight per foot of metal is $w= 2(w_2 + w_3 + w_4 + w_0+200) = 2(395+32.1+198+93-200)= 1036.2$ pounds.

Considering the number of panels of the truss to be 6, the panel length will be 25 feet, and the depth is therefore Cl =0.2018x150=30.2 feet.

The total weight per foot of metal is $w = 2(w_2 + w_3 + w_4 + w_0-200)=2(376+38+201+98.04-200)=1026.08$ pounds.

Finally, by assuming the number of panels to be 7, the panel length is 21.43 feet, the depth is Cl= 0.1846x150= 27.72 feet, andthe total weight per foot of metal is $w=2(w_2 + w_3 + w_4 + w_0-200) = 2(361+37+221+101.4-200)$ = 1040.8 pounds.

From the above computation it is found that 6 panels give the least weight, and therefore they will be adopted as the best design. The depth of the truss is therefore 30.2 feet, but a depth of 30 feet will be used.

2. DETERMINATION OF STRESSES.

The next proceedure in this design was to analyze the stresses in the truss of Fif. 2, taking first the stresses due to dead load. In figuring the stresses for the truss it was assumed that the dead load was concentrated at the lower panel points, and the stresses calculated for one half of the truss only, as the stresses in the other half under the dead load will be the same. The length of span, center to center of pins was considered to be 150 feet, the depth of the truss between centers of trusses, was assumed to be 17,feet.

The length of a panel was taken as 25 feet, as it was found above that six panels will give the best economical arrangement of panel lengths. The angle which the diaganals of the truss make with the vertical was therefore found to be 39° 40'.

The weight per linear foot of trusses and lateral systems was found from the formula w=650+7l=650+7x150=1700 lbs. that of the track was 400 lbs., and that of the stringers and floor

be[illegible] was taken as 60[illegible] lbs. Making [illegible] 7 [illegible] per [illegible].
The dead panel loa[illegible] per truss [illegible] 27[illegible]25/[illegible]=357[illegible]

A sketch is therefore [illegible] follows.

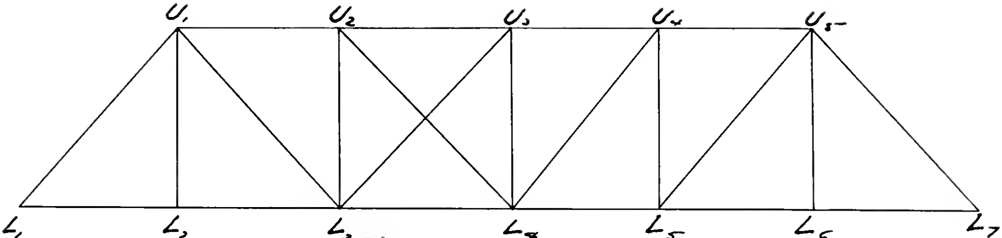


Fig. [illegible]

Dead [illegible].

The shear in [illegible] $U_2 L_3$ = 1/[illegible]3570= 1357 #

The stress in $U_2 L_4$ is therefore 1357[illegible] 10′= [illegible] lbs.

The shear in panel $L_1 L_2$ = 3/2x3575[illegible] #

The stress in $U_1 L_3$ is therefore [illegible] 10′= 7[illegible].

The shear in panel $U_1 L_1$ = 5/2x35750= [illegible]437 #

The stress in $U_1 L_1$ is therefore 84[illegible]75 [illegible] ′ = 109[illegible].

The stress in suspender $U_1 L_2$ equal [illegible]

[illegible] 3575[illegible] lbs.

Next passing [illegible] section [illegible] L_1 [illegible] L_2 [illegible]

the center of [illegible] at L_1 [illegible] M= 0=[illegible]3575[illegible]/[illegible]= $L_1 L_2$ [illegible].

Therefore the stress [illegible] $L_1 L_2$ is $\frac{[illegible]35750x25}{[illegible]}$= 70[illegible] lbs.

and this is also equal to the stress in $L_2 L_3$.

Passing also [illegible] section through $U_2 U_3$ [illegible] L_4 [illegible] $L_3 L_4$ [illegible]

taking the center [illegible] at L_4, [illegible]

$M=0=5/2 \times 33750 \times 75-33750 \times 50-33750 \times 25+U_2 U_3 \times 30.$

Therefore the stress in $U_2 U_3 = -126560$ lbs.

Collecting Results.

Stress in $U_2 L_4 = +21940$ lbs.

" " $U_2 L_3 = -16875$ lbs.

" " $U_1 L_3 = +72310$ lbs.

" " $U_1 L_2 = -33750$ lbs.

" " $U_1 L_1 = -109690$ lbs.

" " $U_1 U_2 = -112500$ lbs.

" " $U_2 U_3 = -126560$ lbs.

" " $L_1 L_2 = +70300$ lbs.

" " $L_3 L_4 = +112500$ lbs.

Live Load Stresses.

In designing this bridge the live load was considered as consisted of 2-177.5 ton engines, followed by 5000 lbs. per foot of track. The maximum shears and bending moments were computed in all thepanels, and the stresses in every member were therefore determined.

Determination of maximum shear in 1 panel. The maximum shear in any panel occurs when the average load in the panel is equal to the average load span the entire span.

The wheel loads were therefore placed at the ~~paal~~ panel point and tested for the criterion in the following manner.

No. of Wheel	With wheel on the left Aver. load in panel	Aver. load on span	With whell on right Aver. load in panel	Aver. load on span
Try wheel 2	12.5/25	41.5/150	37.5/25	41.5/150
" " 3	37.5/25	427.5/150	62.5/25	427.5/150
" " 4	62.5/25	440/150	87.5/25	440/150
" " 5	87.5/25	452.5/150	112.5/25	452.5/150
" " 6	100/25	462/150	116.25/25	462.5/150
" " 7	91.25/25	450/150	107.5/25	450/150
" " 8	82.5/25	440/150	98.75/25	440/150
" " 9	73.75/25	429.5/150	90/25	429.5/150
" " 10	65/25	422.5/150	77/25	422.5/150
" " 11	45/25	410/150	70/25	410/150
" " 12	53.75/25	406.25/150	78.75/25	406.25/150
" " 13	62.5/25	402.5/150	87.5/25	402.5/150
" " 14	87.5/25	415/150	112.5/25	415/150

From the above table it is seen, that wheel 4, 10, 11, 12, 13 satisfy the criterion.

The maximum shears for all those positions of load is computed and the largest maximum is therefore adopted.

With wheel 4 at the panel point the reaction due to these load is $R = \frac{20455+355 \times 34+85 \times 17}{150} = 226.46$ and the shear at that section is found to be $V = 226.46-24 = 202.46$ kips or 202460 lbs.

With wheel 10 at the panel point the reaction is $R = \frac{9742.5+242.5 \times 72+108 \times 36}{150} = 224.55$ kips and the shear is therefore

$V = 224.55-33.8 = 190.75$ kips $=$ 190750 lbs.

With wheel 11 at the panel point, the reaction is found to be $R = \frac{7321.25+210 \times 80+200 \times 40}{150} = 214.14$ kips and the shear is therefore $V = 214.14-28.05 = 186.09$ kips or 186090 lbs.

With wheel 12 at the panel point, the reaction is $R = \frac{6248.75+193.75 \times 85+212.5 \times 42.5}{150} = 211.66$ kips and the shear is therefore found to be $V = 211.66-25.15 = 186.51$ kips $=$ 186510 lbs.

Finally with wheel 13 at the panel point the reaction is

$R = \dfrac{5257.5+177.5 \times 90+225 \times 45}{150} = 209.05$ kips and the shear due to these loads $V = 209.05-24 = 185.05$ kips or 185050 lbs.

From the above figures it is seen that wheel 4 produces the maximum shear in the first panel, and its amount is found to be equal to 202460 lbs.

Determination of maximum shear in 2nd panel.

In order to find the maximum shear in this panel it is necessary to place the wheel at the panel point and test them for the criterion in the following manner.

No. of wheel	With whell on left: Aver. load in panel	With whell on left: Aver. load on span	With wheel on right: Aver. load in panel	With wheel on right: Aver. load on span
Try wheel 2	12.5/25	355/150	37.5/25	350/150
" " 3	37.5/25	365/150	62.5/25	365/150
" " 4	62.5/25	377.5/150	87.5/25	377.5/150
" " 5	87.5/25	390/150	112.5/25	390/150
" " 6	100/25	412.5/150	116.25/25	412.5/150
" " 7	91.25/25	425/150	107.5/25	412.5/150
" " 8	82.5/25	440/150	98.75/50	440/150
" " 9	73.75/25	452.5/150	30/50	452.5/150
" " 10	65/25	460/150	77.5/25	460/150
" " 11	45/25	430/150	70/25	420/150
" " 12	53/25	417.5/150	78/25	417.5/150
" " 13	62.5/25	405/150	87.5/25	405/150

From the above table it was found that wheel 4, 9, 10, 12 satisfy the criterion. By comparing their shears the largest one will be selected.

With wheel 4 at thepanel point the reaction is found to be $R = \dfrac{20455+355 \times 9+22.5 \times 4.5}{150} = 158.35$ kips and the shear is therefore $V = 158.35-24 = 134.35$ kips or 134350 lbs.

With wheel 9 at the panel point the reaction is $R = \dfrac{20455+355 \times 39+97.5 \times 19.5}{150} = 241.34$ kips andthe shear due to these loads

V= 241.34-37.5-45.8= 108.04 kips.

With wheel 10 at the panel point the reaction is found to be $R= \frac{19092.5+342.5\times47+117.5\times23.5}{150}$= 253 kips and the shear is therefore V= 253-100-41.6= 111.4 kips or 111400 lbs.

Finally with wheel 12 at thepanel point thereaction is found to be $R= \frac{11892.5+267.5\times60+150\times30}{150}$= 216.28 kips and the reaction is therefore V= 216.28-73.75-25.15= 119.38 kips

From theabove it is seen that wheel 4 produces the maximum shear in the second panel and its amount is equivalent to 134350 lbs.

Determination of maximum shear in 3rd panel.

The method of proceedure of finding the maximum shear in this panel is obtained in the same way, as in the previous cases, by placing the wheel load at the panel points in the following manner.

No.of wheel	With wheel on left of panel point		With wheel on right of panel point	
	Aver.load in panel	Aver.load on span	Aver.load in panel	Aver.load on span
Try wheel 2	12.5/25	290/150	37.5/25	290/150
" " 3	37.5/25	306.25/150	62.5/25	306.25/150
" " 4	62.5/25	322.5/150	87.5/25	322.5/150
" " 5	87.5/25	322.5/150	112.5/25	322.5/150
" " 6	100/25	355/150	116.25/25	355/150
" " 7	91.25/25	362/150	108.5/25	362/150
" " 8	82.25/25	377/150	98.75/25	377/150
" " 9	73.75/25	389.5/150	90/25	389.5/150
" " 10	65/25	410/150	77.5/25	410/150
" " 11	45/25	430/150	70/25	430/150
" " 12	53.75/25	442.5/150	98.75/25	442.5/150
" " 13	62.5/25	455/150	87.5/25	455/150

From the above figures it was found that wheel 3, 10, 12, 13 satisfy the criterion.

With wheel 3 at the panel point thereaction is R=

R= 13520/150= 90.13 kips or 90130 lbs. and therefore the shear is equal V= 90.13-11.5- 78.63 kips or, 78630 lbs.

With wheel 10 at the panel point the reaction is found to be $R= \frac{20455+355x22+55x11}{150}=$ 192.46 kips and the shear is V= 192.46-112.5-41.6= 38.36 kips.

With wheel 12 at the panel point the reaction is $R= \frac{20455+355x35+87.5x17.5}{150}=$ 229.4 kips andthe shear is therefore

V= 229.4-161.25-25.11- 43.04 kips.

With wheel 13 at the panel point the reaction is found to be $R= \frac{20455+355x40+100x20}{150}=$ 244.36 kips and the shear due to these loads is V= 244.36-177.5-24= 42.86 kips.

By comparing the above values it is found that wheel 3 produces themaximum shear in this panel and its value is equal to 78630 lbs.

Determination of maximum shear in 4th panel.

In a similar manner it was found that wheel 2 produces a maximum shear in this panel. Thereaction is therefore $R= \frac{5790+190x2}{150}=$ 41.13 kips, and the shear is found to be V= 41.13-4= 37.13 kips or 37130 lbs.

It was also found that wheel 2 produces a maximum shear in 5th panel. The reaction is $R= \frac{2050+128.75x1}{150}=$ 14.52 kips. and the shear is therefore V= 14.52-4-10.52 kips or 10520 lbs.

Determination of maximum bending moment in 1st panel.

The maximum moment in any panel occurs when the average load upon the span is equal or just great than the average load in front of the panel point.

The wheel loads were placed at the panel point and tested for the criterion in the following manner.

No. of wheel	With wheel load on left of panel point: Aver. load in front of panel	Aver. load on the span	With wheel load on right of panel point: Aver. load in front of panel	Aver. load on the span
Try wheel 2	12.5/25	415/150	37.5/25	415/150
" " 3	37.5/25	427.5/150	62.5/25	427.5/150
" " 4	62.5/25	440/150	87.5/25	440/150
" " 5	87.5/25	452.5/150	112.5/25	452.5/150
" " 6	100/25	462.5/150	116.25/25	462.5/150
" " 7	91.25/25	450/150	107.5/25	450/150
" " 8	82.5/25	440/150	98.75/25	440/150
" " 9	73.75/25	429.5/150	90/25	429.5/150
" " 10	65/25	427.5/150	77/25	422.5/150
" " 11	45/25	410/150	70/25	410/150
" " 12	53.75/25	406.25/150	78.75/25	406.25/150
" " 13	62.5/25	402.5/150	87.5/25	402.5/150
" " 14	87.5/25	415/150	112.5/25	415/150

From the above table it was found that wheel 4, 10, 11 12, 13 satisfy the criterion.

With wheel 4 at the panel point thereaction due to these loads is $R = \frac{20455+355\times34+85\times17}{150} = 226.46$ and the maximum bending moment is therefore $M = 226.46\times25-600 = 5061500$ ft lbs.

With wheel 10 athe panel point the reaction is $R = \frac{9742.5+242.5\times72+180\times36}{150} = 224.55$ kips and the maximum bending moment is therefore $M = 224.55\times25-845 = 4768750$ ft lbs.

With wheel 12 at the panel point the reaction is $R = \frac{7321.25+210\times80+200\times40}{150} - 214.14$. The maximum bending moment is therefore $M = 214.14\times25 - 701.25 = 465252P$ ft lbs.

With wheel 12 at the panel point the reaction is $R = \frac{6248.75+193.75\times85+212.5\times42.5}{150} = 211.66$ kips.

The maximum bending moment is there M= 211.66x25-628.75= 4662750 ft-lbs.

Finally with wheel 13 at the panel point the reaction was found to be R= $\frac{5257.5+177.5x90+22.5x45}{150}$= 209.05. kips

The maximum bending moment is therefore M= 209.05x25-600= 4626250 ft lbs.

From the above mentioned it is seen that wheel 4 produces the largest bending moment and its amount equals 5061500 foot-lbs.

Determination of maximum bending moment in 2nd panel.

In this case the wheel load are also placed at the panel point and tested for the criterion in the following manner.

Try wheel	2	12.5/50	355/150	37.5/50	355/150
" "	3	37.5/50	365/150	62.5/50	365/150
" "	4	62.5/50	377.5/150	87.5/50	377.5/150
" "	5	87.5/50	390/150	112.5/50	390/150
" "	6	112.5/50	412.5/150	128.75/50	412.5/150
" "	7	128.75/50	425/150	145/50	425/150
" "	11	127.5/50	430/150	152.5/50	430/150
" Q	12	127.5/50	417.5/150	152.5/50	417.5/150
" "	13	127.5/50	405/150	152.5/50	405/150

From the table it is seen that wheel 7, 11, 12 and 13 satisfy the criterion.

With wheel 7 at the panel point the reaction is R= $\frac{20455+355x28+70x14}{150}$= 209.16 kips, and the maximum bending moment is therefore M= 208.16x50-2683.75=7874250 ft-lbs.

With wheel 11 at the panel point, the reaction is found to be R- $\frac{14167.5+292.5x55+137.50x27.5}{150}$= 226.9 kips

The maximum bending moment is therefore M= 226.9x50-3835-7510000 ft-lbs.

With wheel 12 at the panel point, the reaction is

$R= \frac{11892.5+267.5x60+150x30}{150}$= 216.28 Kips, and the maximum bending is M= 216.28x50-3322.5= 7491500 ft-lbs.

Finally with wheel 13 at the panel point thereaction is

$R= \frac{9742.5+242.5x65+162.5x32.5}{150}$= 205.3 Kips, and the maximum bending moment is M= 205.2x50-2810= 7450000ft-lbs.

By comparing the above figures it is found that wheel 7 produces the largest bending moment and its amount equals 7874250 foot-lbs.

Determination of maximum bending moment in 3rd panel.

In this case the wheel loads are also placed at the panel point and tested for the criterion in the following manner.

No. of wheel	With wheel on left of panel point		With wheel on right of panel point	
	Aver.load in panel	Aver.load on span	Aver.load in panel	Aver.load on span
Try wheel 2	12.5/75	290/150	37.5/75	290/150
" " 3	37.5/75	306.25/150	62.5/75	306.25/150
" " 4	62.5/75	322.25/150	87.5/75	322.25/150
" " 5	87.5/75	322.25/150	112.5/75	322.25/150
" " 11	127.5/75	367.5/150	152.5/75	367.5/150
" " 12	127.5/75	355/150	152.5/75	355/150
" " 13	127.5/75	342.5/150	152.5/75	342.5/150
" " 14	152.5/75	355/150	177.5/75	355/150

From the table it is found that wheel 12 and 13 satisfy the criterion.

With wheel 12 at the panel point the reaction is R=

$\frac{11892.5+267.5x35+87.5x17.5}{150}$= 151.9 Kips, and thebending moment is therefore M= 151.9x75-3322.5= 8070000 ft-lbs.

With wheel 13 at the panel point the reaction is R=

$\frac{3742.5+242.5x45+112.5x22.5}{150}$= 154.1 Kips, and the maximum bending

moment is therefore M= 154.1x75-3572.5= 798500 ft-lbs.

By comparing the last figures it is found that wheel 12 produces the largest bending moment and its amount equals 8070000 foot-lbs.

After finding the shears and bending moment in all the panel, the stresses in web numbers and also in the chords are obtained in the following manner.

The stress in $U_1 L_1$= 202460 Sec 39°40′= 253200#
" " " $U_1 L_3$= 134350 Sec 39°40′= 174650#
" " " $U_2 L_4$= 78630 Sec 39°40′= 102200#
" " " $U_1 L_2$= 94550#
" " " $U_2 L_3$= 78630#
" " " $U_3 L_4$= 37130#
" " " $U_1 U_2$= 7874250/30= 262475#
" " " $U_2 U_3$= 8070000/30= 269000#
" " " $L_1 L_2$= 5061500/30= 168720#
" " " $L_3 L_4$= 7874250/30= 262475#

The stress in the counter brace $U_3 L_5$ is found by considering the live and dead load shear in the panel $U_3 L_5$.

The live load shear in this panel is 37130, which evidently causes compression in $U_4 L_4$. But the dead load shear in this panel is- 16875 that produces tension in the above member. The resultant is 20255 which is the vertical component of $U_3 L_5$ causing compression in $U_4 L_4$ and therefore a counter brace is needed.

The stress in $U_3 L_5$ is therefore 20255 Sec 39°40′= 26330#.

With regard to a counter stress in panel $U_4 L_6$ the dead

and live shears were also considered.

The live load is + 10520#, producing compression in U_5L_5, but the dead load shear is - 55625, causing tension in the above member. The resultant is - 45105, producing tension in the diaganal U_5L_5 and therefore no counter brace is needed in panel U_4L_6.

Determination of maximum shear and bending moment in a 25-0 stringer.

The maximum shear in a stringer occurs at the point of supports.

The wheel loads were placed at the end and the shears found in the following manner.

With wheel 1 at the end the reaction is found to be

$R= \frac{1037.5+112.5x2}{25} =$ 50.5 Kips or 50500 lbs.

With wheel 2 at the end the reaction is $R= \frac{1650+116.25x1}{25} =$ 70.65 Kips= 70650 lbs.

With wheel 3 at the end the reaction is therefore $R= \frac{1506.25+107.5x1}{25} =$ 64.5 Kips= 64500 lbs.

Placing wheel 4 at the end the reaction is found to be R= 1401.25/25= 56.05 Kips= 56050 lbs.

Finally with wheel 5 at the end thereaction is R= 1145/25= 45.8 Kips= 45800 lbs.

The maximum shear therefore occurs under wheel 2 and its amount is 70650 lbs.

The condition for determining the maximum bending moment in a stringer is the same so that for a simple beam. The criterion is, that the wheel producing the maximum bending

moment must be so far from the end, as the center of gravity of all the wheels is from the other end. For this purpose the wheel is placed at the center of the stringer and tested for the criterion in the following manner.

No. of wheel	With wheel on left of panel point		With wheel on right of panel point	
	Aver. load in panel	Aver. load on span	Aver. load in panel	Aver. load on span
Try wheel 2	12.5/12.5	87.5/25	37.5/12.5	87.5/25
" " 3	25/125	100/25	50/12.5	100/25
" " 4	50/12.5	100/25	75/12.5	100/25
" " 5	50/12.5	91.25/25	75/12.5	91.25/25

From theabove table it is found that wheel 3 and 4 satisfy the criterion.

With wheel 3 at the center, the reaction is found to be $R = \frac{750+100x3.5}{25} = 44$ Kips= 44000 lbs., and the maximum bending moment is M= 44000x11.25-12500= 370000 ft-lbs.

With wheel 4 at the center the reaction is R= $\frac{750+100x6.25}{25} = 55$ Kips= 55000 lbs.

The maximum bending moment is therefore M= 55000x13.75-37500= 381250 ft-lbs.

From the last expression it is seen that wheel 4 produces the largest bending moment and its moment is 381250 foot-lbs.

Determination of maximum floor beam concentration.

The criterion for maximum floor beam reaction is that the average load in the panel on the left must be equal to or just greater than the average panel load upon the right including the wheel at the panel point. The wheel loads are placed at the panel point and tested for the criterion in the following manner.

No. of wheel	With wheel on left of panel point: Aver. load on left	With wheel on left of panel point: Aver. load on span	With wheel on right of panel point: Aver. load on left	With wheel on right of panel point: Aver. load on span
Try wheel 2	12.5/25	116.25/25	37.5/25	91.25/25
" " 3	37.5/25	107.5/25	62.5/25	82.5/25
" " 4	62.5/25	98.75/25	87.5/25	73.75/25
" " 5	87.5/25	90/25	112.5/25	65/25

From above figures it is found that wheel 4 and 5 produces the maximum floor beam reaction.

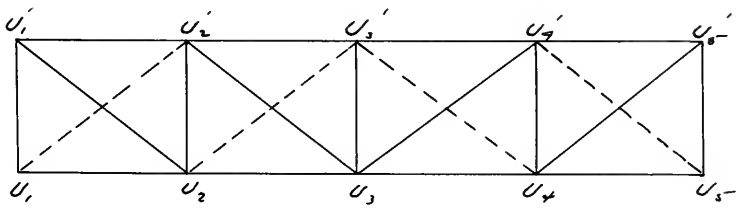
With wheel 5 at the center the maximum floor beam reaction is found to be $R=R_1+R_2=114.5/25+\frac{975+87.5\times2}{25}=91.8$ Kips= 91800 lbs.

With wheel 4 at the center the reaction is $R=R_1+R_2=\frac{1401.25}{25}-525+62.5\times7/25=94.55$ Kips= 94550 lbs.

Therefore wheel 4 produces the maximum floor beam reaction and its amount is 94550 lbs.

Determination of Stress in the top lateral bracing.

These laterals will be proportioned to resist a lateral force of 200 pounds for each linear foot of the span, thus making the panel load equal to 200x25= 5000 pounds. A sketch of the laterals is shown below and with the above loading the stresses are determined as follows.



The angle that the diaganals make with the vertical is $55^\circ 50'$.

The stress in $U_2'U_3$= 5000/2 Sec 55°50'= 4500 pounds.

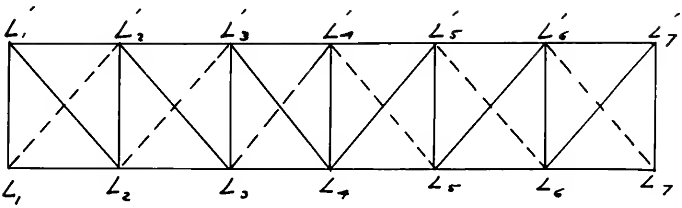
" " " $U_1'U_2$= 3/2x5000 Sec° 55' 50= 13500 "

" " " $U_2'U_2$= 2500 pounds.

" " " $U_1'U_1$= 7500 "

The bottom lateral bracing is proportioned to resist a lateral force of 600 pounds for each foot of the span; 450 pounds of this to be treated as a moving load, and as acting on a train of cars, at a line of 6 feet above base of rail.

A sketch is also given below and the stresses determined as follows:



Fixed load stresses.

In this case the fixed load stresses will be determined first. The fixed panel load is 150x25= 3750 pounds. With this loading the stresses are figured in the following manner.

The stress in $L_1'L_1$= 5/2x3750= 9375 pounds.

" " " $L_1'L_2$= 9375x1.78= 16800 "

" " " $L_1'L_2$= 3/2x3750= 5620 "

" " " $L_2'L_3$= 5620x1.78= 9960 "

" " " $L_3'L_3$= 3750/2= 1875 "

" " " $L_3'L_4$= 1875x1.78= 3380 "

" " " $L_3'L_4'$= $\frac{3750x150x150}{8x17}$= 24880 pounds.

" " " $L_2'L_3'$= $\frac{Ax3750x25}{17}$=-22060 "

The stress in $L_1' L_2' = \frac{5x3750x25}{2x17} =$ − 13760 pounds.

" " " $L_2 L_3 =$ + 22060 "

" " " $L_1 L_2 =$ + 13760 "

Live load stresses.

The live load panel load is 450x25= 11250 pounds.

The shear in panel $L_3 L_4 =$ 11250 pounds.

The stress in $L_3' L_4 =$ 11250x1.78= 19980 pounds.

The shear in panel $L_2 L_3 =$ 10/6x11250= 18760 "

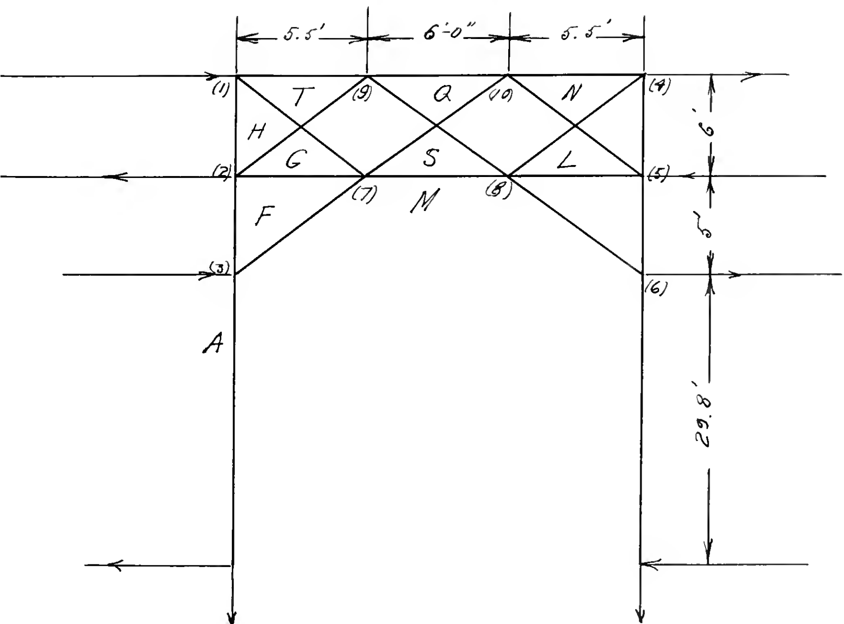
The stress in $L_2' L_3$ = 18760x1.78= 33200 "

The shear in panel $L_1 L_2 =$ 15/6x11250= 28120 "

The stress in $L_1' L_2 =$ 28120x1.78= 49760 "

Determination of Stresses in the Portal Bracing.

A sketch and dimensions of the Portal Bracing is given below and the stress are determined in the following manner.



The bracing was design to resist a lateral force of 150 pounds per. linear foot of spans. The wind load per panel is therefore 150x25= 3750 pounds. The force applied at the top of the portal is 2x3750= 7500 pounds. With this loading and also with imaginary forces that are put in for convenient coputation the stresses are figured as follows.

The reaction is found by taking moments about the base of the Portal, and $R = \frac{7500 \times 39.07}{17} = 17200$ pounds.

Let the shear be divided equally between the two diagonals cut by any section parallel to the end post. The stress in each diagonal is 8600x1.41= 12040 pounds. The imaginary force at joint (2) is 12040 cos 4730= - 8120 pounds.

Taking moments about joint (3), the imaginary force at (1) is found to be $M=0=P\times11+8120+3750\times29.85$, and P=-13920 pounds. Also taking moments about joint (2), the horizontal force at (3), is found to be $M=0=P\times5+3750\times34.8-13920\times6$ P= 9550 pounds.

Taking out joint (3) by itself and solving horizontally we have, $\Sigma y = 0 = 9550 + FM \cos 42^\circ 20'$; FM=13370 pounds also solving vertically it becomes $\Sigma y = 0 = AF + 13370 \sin 42^\circ 20' - 17200$, hence AF= 8240 pounds.

Taking out joint (2) by itself and solving horizontally, we have $\Sigma x = 0 = AH + 12040 \sin 47^\circ 30' - 8240$, hence AH=-670 pounds.

By taking out joint (1) by itself and solving also vertically, we get $\Sigma y = 0 = AH - HT \sin 47^\circ 30'$. Therefore HT=+900 lbs.

By solving it horizontally, we have, $0 = DT + 900 \cos 47^\circ 30' + 21420$ Hence DT=-22025 pounds.

Passing a section through SQ, QS, SM andtaking moments

about joint (7), we have, $M=0=DQ\times 6-1204\times 6 \sin 45^\circ+21420\times 6+9550\times 5+3750\times 34.8-17200\times 5.5$.

Therefore DQ=-26930 pounds.

Next taking out joint (10) by itself and solving horizontally, we have, $\Sigma x=0=-DQ+DN-12040\cos 45^\circ-12040\cos 45^\circ$

Hence D N=+22386 pounds.

Taking also joint 4 by itself and solving horizontally, we have $\Sigma x=0=-DN-NL\cos 47^\circ 30'$

Therefore NL=-31340 pounds.

Next taking out joint 5 by itself and solving horizontally, we have $\Sigma x=0=-LK+12040\cos 47^\circ 30'$.

Hence LK=+16856 pounds.

Finally, taking out joint (8) by itself and solving horizontally, we have, $\Sigma x=0=-MS+12040\cos 45^\circ+LK+MK\cos 47^\circ 30'-8120$.

Hence MS=+4150 pounds, and s lving it vertically, we get $\Sigma y=0=12040\cos 45^\circ-31340\cos 47^\circ 30'+MK\cos 42^\circ 30'$.

Therefore MK=+16760 pounds.

3. Calculations of Sections and Weights.

Design of floor timbers.

The greatest stress in the cross-tie is produced by the loading of 25000 pounds placed on one axle. If the cross-ties be 8 inches wide and spaced 6 inches in the clear, three ties and spaces will cover a length of 3 1/2 feet. Assuming the total weight of the track as 450 pounds per linear foot, the weight for a length of 3 1/2 feet is 1575 pounds, and for each rail on each tie 8600 pounds. The dead load is relatively so small that it may be assumed to be also concentrated at the

track rails, without appreciable error. The stringers are spaced 6 1/2 feet apart. The cross-tie is a beam with two concentrated loads, each of 8600 pounds, spaced 4 feet 11 1/2 inches apart and placed symmetrically with respect to the supports furnished by the stringers. The bending moment is therefore 8600x9.25= 79550 inch-pounds. For a unit stress of 1000 pounds per square inch and a width of 8 inches, the required depth of the cross-tie is found to be 8 inches. The bearing value of timber is taken as 250 pounds per square inch. The bearing area required is then 8600/250=34 square inches and if the width of thebase of the rail be 6 inches the breadth must be 7 inches which is safe enough within the value assumed.

Design of Track Stringers.

The span of the stringer equals the panel length of the truss, or 25 feet. Let the weight of the track carried by one stringer be assumed to be 5000 pounds, making the dead load 10500 pounds.

The maximum shear for a stringer of 25 feet was found to be 70650 pounds, while the dead load shear is 5250 pounds, making the total shear 75900 pounds.

Let thedepth of the stringer be taken as 36 inches. A thickness of 7/16 allows for enough rivets to be deducted from the web. But this thickness, however, required stiffeners to be used, which may be avoided by increasing the thickness to 1/2.

The dead load bending moment is $\frac{10500\text{x}25}{8}$= 33100 ft-lbs.,

while the live load bending moment was found to be 381250 ft-lbs., thus making thetotal moment 381250+33100= 414350 foot-pounds.

Assuming the unit tensile stress as 10000 pounds, and the effective depth to be 32.5 inches, one half of flange is therefore $A = \frac{414350 \times 12}{32.5 \times 10000} = 15.3$ square inches.

Let $2L^s - 6'' \times 6'' \times 3/4''$ be assumed, thenet area is A= 2(8.44-.75)= 15.38 square inches.

The actual effective depth is 32.44 inches, and the revised flange area is $A = \frac{414350 \times 12}{32.44 \times 10000} = 15.32$ square inches, and therefore these angles will be used.

Let the rivet pitch in the flange be determined next. The maximum vertical shear at the end is 75900 pounds, and the increment of flange stress per linear inch is $\frac{15.32}{15.38} \times \frac{75900}{32.44} = 2340$ pounds.

The vertical load on the flange is 25600/42= 595 pounds.

The resultant of these horizontal and vertical components is $R = (2340^2 + 595^2)^{\frac{1}{2}} = 2410$ pounds.

The allowable bearing value of 7/8" rivet in 1/2" web plate is 8/10x7/8x1/2x1500= 5280 pounds, and hence the theoretic rivet pitch at the end is 5280/2410= 2.3 inches.

Since the vertical angles which connect the end of the stringer to theweb of the floor beam are to be straight, fillers whose thickness equals that of the flange angles are placed. The value of 7/8" rivet in single shear at 7200 pounds per square inch is 4320 pounds.

The number of rivets required to transmit the shear

from the web of the stringer to the connection angles is 75900/5250= 15.

The rivets connecting the other legs of these angles to the web of the floor beam are field rivets, and since they are in single shear thenumber required is 75900/4320= 18 shop rivets or 28 field rivets.

The estimate of the weight of one stringer is as follows:

4 flange angles 6x6x3/4x24'-11"	@	28.7#=	2870#
1 web plate 36x1/2x24'-11"	@	61.2#=	1530#
4 connection angles 6x6x1/2x3'-4" 3/8		19.6#=	274.4#
4 fillers 9x1/2x2'-0"	@	15.3#=	122.4#
250 pairs of rivet heads		0.369#=	88.0#
Total..............................			4884.8#

a quantity nearly equal to the value assumed.

Design of Floor Beams.

For convenience in erection, bracket angles are riveted to the lower flange of the floor beam or to the web just above the flange, on which to support the stringers until their end connecting angles are riveted to the floor-beam webs.

Assuming that the vertical legs of the flange angles do not exceed 4 inches, it is found that a depth of 48 inches will bring the top of the cross-ties about 3 inches higher than the top of the floor beam.

The floor beam carries, in addition to its own weight, two concentrated loads 3'-3" from its center, each load consisting of the maximum sum of the adjacent reactions of the

stringers on both sides. This sum includes the weight of one stringer, and of the track which it supports, and the corresponding live load. The maximum floor beam reaction was found to be 94550 pounds, and assuming the weight of the floor beam to be 5000 pounds, the total maximum shear is therefore 94550+10500+2500= 107550 pounds, where 10500 is the dead load reaction of one stringer.

The allowable unit stress is 8/10x9000- 7200#, and the required net area of the web is 107550/7200= 14.93 square inches.

Let the thickness of the web be taken as 7/16", then the gross area is 48x7/16= 20.66 square inches, which allows enough rivets to be deducted from the web ~~from~~ for the splice section. The length of the floor beam is 17 feet, and the dead load bending moment is $M = \frac{5000 \times 17}{8} = 10625$ foot-lbs.
The live load bending moment was found to be 551512.5 foot-lbs., making the total bending moment 562137.5 foot-lbs.

Assuming the effective depth to be 46 inches, the area of one half of the flange is $A = \frac{562137.5 \times 12}{10000 \times 46} =$ 14.61 square inches.

Let the following composition of the flange be taken, which furnishes a net area of 14.74 square inches. 2 angles, 5x4x9/16, 2(4.75-1.10)= 7.24 square inches.

1 cover plate, 12x3/4, 9-1.5- 7.50

Total net area..............14.74 square inches.

The center of gravity of the solid section of the upper flange is $\frac{7.24 \times 1.48}{14.74} =$ 0.73 inches, below the backs of the angles, and that of the net section of the lower flange is

9.5x1.48/18.5= 0.77 inches above the backs of the angles. The effective depth is therefore 48-1.5= 46.5 inches, and the revised flange area is $A = \frac{6745650}{10000 \times 46.5} = 14.59$ square inches.

The above mentioned angles a plate mill therefore be used.

Let the rivet pitch of the flange angles be considered next.

The bearing value of 7/8" rivet in 7/16" web plate is 8/10x7/8x7/16x15000= 4620#.

The increment of flange stress per linear inch is $\frac{14.59\text{x}107550}{14.74\quad 4659}$= 2100#, and the pitch is 4620/2100= 2.45 inches.

In the space between the stringers the pitch is made 6 inches, the maximum allowed.

In figuring the number of rivets in the web splice the bearing value of rivets will be considered, since their value is less than that in double shear. The bearing value of 7/8" rivet in a 7/16" web plate is 8/10x7/8x7/10x15000= 4520#.

The number of rivets is therefore 107550/4620= 24 (shop rivets).

Since the rivets are field ones, their number will be increased by 50% and is therefore 1.5x24= 36.

The estimate of the weight of one floor beam is as follows:

2 flange angles, 5x5x3/16x16'-1"	@ 16.2#	523	pounds
2 flange angles, 5x4x3/6x14x4	" 16.2#	464	"
1 cover plate, 12x3/4x16'-1"	" 30.6#	490	"
1 cover plate, 12x3/4x11'-4"	" 30.6#	350	"
1 web plate, 48x7/16x11'-1 1/2"	" 71.44#	785	"
2 web plates, 23x7/16x7'-5 1/2"	" 34.24#	250	"
4 splice plates, 30x3/16x3'-8"	" 57.4#	175	"
4 filler plates, 27x1/2x2'-10"	" 45.9#	95	"
4 connection angles, 3 1/2x 3 1/2x1/2x2'-4"	11.1#	25	"

4 connection angles, 3 1/2x3 1/2x 1/2x2'-11"	@ 11.5#....22	pounds
4 filler plates, 7x3/16x2'-11"	" 13.4#....28	"
4 angles, 4x3 1/2x3/8x2'-7 1/2"	" 3.1#....20	"
4 filler plates, 3 1/2x3/16x2'-1"	" 6.7#....15	"
4 filler plates, 3 1/2x1/2x2'-4"	" 6.7#....15	"
4 angles, 3 1/2x3 1/2x 3/8x3'-3"	" 8.5#...111	"
4 bracket angles, 5x4x3/8x1'-3"	" 85# 55	"
600 airs of rivet heads	" 0.369#..225	"
Total..................................	4647	"

Sections of Intermediate Posts.

Let be required to design the section of the post U_2L_3 Neglecting the wind stresses, which are relatively too small to affect, the total stress to be considered is 78630+16875/2= 87060. Let 2-12 inch- 35# channels be tried. The radius of gyration is 4.17" and the allowable unit stress is $P= 8500-\frac{.45x30x12}{4.17}=$ 4620 pounds.

The required area is 87060/4620= 18.86 square inches. The area of the two channels is 2x10.29= 20.58 square inches, and therefore they will be used. The distance back to back of channels, in order to make the radü of gyration equal is 9.59.

In a similar manner it is found that two 12- inch 20.5 pounds channels are needed for the post U_3L_4 the radius of gyration being 4.61 inches, the allowable unit compressive stress per square inch in $P= 8500- \frac{45x30x12}{4.61}=$ 3260#.

The required sectional area is 37130/3260= 11.4 square inches. The area of the two channesl is 2x6.03= 12.06 sq. inches and therefore they will be used. The distance back to back of channels is found to be 10.48 inches.

Sections of Diagonals and Suspender.

Since the stress in U_1L_3 is a tension of 210800 pounds, it may be)composed of one pair of eye-bars. The wind stress may be neglected in designing the member according to the specifications. For the unit tensile stress of 10000pounds per square inch, the sectional area must be 210800/10000= 21.08 square inches. Two eye-bars, 8"x1 3/8", provide an area of 22 square inches, and therefore they will be used.

The stress in U_2L_4 was found to be 113170 pounds, and for a unit tensile stress of 10000 pounds per square inch the required sectional area is 113170/10000= 11.315 square inches.

Two eye-bars, 6"x1", provide an area of 12 square inches and therefore they will be used.

In a similar manner the counter brace is designed. The stress in U_3L_3 is a tension of 26330 pounds. The required area is 26330/10000=2.63 square inches. Therefore one eye-bar, 4'x3/4", which provides a sectional area of 3 square inches will be used.

The suspender, U_1L_2, will be designed as a stiff member. It required net sectional area is 111320/8000= 14.92 square inches. Two 12- inch 35 pound channels will be selected, as they will furnish 15.5 square inches, after deducting two rivet holes in both the web and flanges of each channel.

Lower Chord Sections.

Since the wind stresses in those chords are less than 30 percent of the maximum strains due to the dead and live loads, they can be therefore neglected.

The equivalent live load is $L_1 L_2$ is 203870 pounds. With a unit stress of 10000 pounds per square inch the required area is 203870/10000= 20.39 square inches. Let $L_1 L_2$ be composed of two web plates and f ur angles. Since the eye-bar heads of the 8 inch eye-bars are 17 inches deep, let the web plates be made 18 inches deep so as to avoid cutting the angles in order to pass the eye-bar heads at L_3. Selecting 2 web plates 18"x1/2" and 4 angles 3 1/2"x3 1/2"x 5/8", the rivets in the end pin plates can be so arranged as not to deduct more than 3 rivet holes in each web plate and o e in each angle, giving a net area of 20.46 square inches.

The area of 2 web plates, 18"x1/2" is 18 square inches, while the area of 4 angles, 3 1/2"x3 1/2"x5/8" is 7.96 square inches, making the total gross area 25.96 square inches. The area of 4 rivet holes in the angles is 4x5/8- 2.5 square inches, while the area of 6 rivet holes in the webs is 6"x1/2"= 3 square inches. The net area is therefore 25.96-2.5-3= 20.46 square inches, and therefore this section will be used.

The member $L_3 L_4$ will consist of eye-bars. The stress in this member is 318725 pounds, and ~~will~~ with an allowable unit stress of 10000 pounds per square inch, the required net area is 318725/10000= 31.87 square inches. Let 4 eye-bars, 8"x1", be selected. The area of these eye-bars is 4x8x1= 32 square inches, and therefore they will be used.

Upper Chord Sections.

Let the chord $U_1 U_2$ be designed first.

The equivalent live load stress in this member is 318725 lbs.

Let the composition of the section be as follows:

1 cover plate, 26"x3/8"...............9.75 square inches
4 angles, 3 1/2"x3 1/2"x3/8"...........9.92 " "
2 web plates, 18"x7/16"...............15.76 " "
2 plats, 5"x3/16".....................5.60 " "
Total.....................41.03

This section will now be investigated in order to determine if it fulfills the conditions, and does not give an excess or deficiency of area. The center of gravity is computed by taking moments about an axis through the center of the top plate and parallel to its width. It was best to arrange the principal quantities in tabular form, A representing the area of any part in square inches, and l its lever arm in inches with respect to the axis mentioned above.

Piece	A	l	Al
1 cover plate	9.75	0	0
2 angles	4.96	1.2	5.95
2 angles	4.96	17.18	85.21
2 web plates	15.76	9.19	144.83
2 plates	5.60	18.47	103.43
Sums	41.03		339.42

Then the distance from center of cover plate to the center of gravity of section is $g = Al/A = 339.42/41.03 = 8.3$ inches, and and the eccentricity of the section, or distance from center of webs to neutral axis is $e = 9.18-8.3 = 0.88$ inches. The moment of inertio of the section is now computed, neglecting the

moments of inertia of the plates about their own axis parallel to their width; thus

Piece	A	I′	h	Ah^2
1 cover plate	9.75	.20	9.19	823.39
4 angles	9.92	11.48	7.99	633.30
2 web plates	15.76	425.20	0	0
2 flats	5.6	.40	9.25	479.13
Sums	41.03	437.28		1935.82

whence I= $(I'-AL^2)$= 2372.1 inches, and the radius of gyration of the section is r= $(2373.1/41.03)^{\frac{1}{2}}$ ~~1/2~~= 7.58 inches.

Lastly, by the column formula of the Specifications.

P= 10000-45x30x12/7.58= 7860 pounds. per square inch. As the stress in the chord is 318725 pounds, the area required is 318725/7860= 40.5 square inches, and therefore the assumed section will be adopted.

The chord U_2U_3 will be designed next. The equivalent live load stress in this chord is 332290 pounds.

Let the composition of the section be as follows:

1 cover plate, 26"x3/8"..................... .9.75 square inches
4 angles, 3 1/2"x3 1/2"x7/16"...............11.48 " "
2 web plates 18"x7/16"......................15.76 " "
2 flats, 5"x1/2"........................... 5.00 " "
Total.........................41.99 square inches.

The center of gravity is computed by taking moments about an axis through the center of the top plate and parallel to its width. A table similar to the previous one will be arrange in which represents the area of any part in square inches, and l its lever arm in inches with respect to the arix mentioned

above.

Piece	A	l	Al
1 cover plate	9.75	0	0
2 angles	5.74	1.15	6.60
2 angles	5.74	16.86	97.18
2 web plates	15.76	9.11	143.57
2 flats	5.00	18.61	93.05
Sums	41.99		340.4

g= Al/A= 340.4/41.99= 8.1 inches, and the eccentricity of the section is e = 9.09-8.1= 0.99 inches above the center of the web plate. The moment of inertia of the section is now computed, neglecting the moments of inertia of the plates about their own axis parallel to their width, thus

Piece	A	I'	h	Ah^2
1 cover plate	9.75	0.2	9.19	807.4
4 angles	11.48	13.0	8.08	750.4
2 web plates	15.76	425.2		
2 flats	5.00	0.4	9.25	437.1
Sums	41.99	439.2		1994.9

whence $I = \Sigma(I' + Ah^2)$ =439.2+1994.9= 2434.1 inches^4, and the radius of gyration of the section is r= $(2434.1/41.99)^{\frac{1}{2}}$= 7.6 inches.

Finally, by the column formula of the specifications, P= 10000- 45x30x12/7.6= 7870, and the required area is 318720/7870= 40.5 square inches, which shows that the assumed sectional area can be adopted.

Inspection shows that moments of inertia around the neutral axis parallel to web plates are respectively greater than those computed for the sections of both chord members, and hence

the values of r determined above are the least radii of gyration required in the column formula.

Section of Inclined End Post.

The length of the post is 30xSec 39°40'= 30x1.302= 39,066,feet. Let the composition of the section be as follows:

1 cover plate, 26" x 1/2"..................13 square inches

4 angles, 3 1/2" x 3 1/2" x7/16"............11.48

2 web plates, 18" x /16"..................20.24

2 flats, 5"x1",..........................10.00

Total 54.72 square inches.

This section will now be investigated in order to determine if it fulfills the conditions, and does not give an excess or deficiency of area. The center of gravity is computed by taking moments about an axis through the center of the top plate, and parallel to its width. It was best to arrange the quantities in tabular form, A representing the area of any part in square inches, and l its lever arm in inches with respect to the axis above mentioned.

Piece	A	l	Al
1 cover plate	13.00	0	0
2 angles	5.74	1.04	5.96
2 angles	5.74	17.18	98.61
2 web plates	20.24	9.22	186.61
2 flats	10.00	18.72	187.20
Sums	54.72		478.38

Then the distance from center of cover plate to the center of gravity of section is g= Al/A= 478.38/54.72= 8.74 inches, and the eccentricity of the section is e =9.22-8.74= 0.68 inches. The moment of inertia of the section is now computed, neglecting the moments of inertia of the plates about their own axis parallel to their width; thus

Piece	A	I	h	Ah^2
1 cover plate	13.00	0.2	9.345	1124.06
4 angles	11.48	13.0	8.085	750.4
2 web plates	20.24	546.7		
2 flats	10.00	0.8	9.75	926.4
Sums	54.72	560.7		2800.9

whence I= 560.7+2800.9= 3361.6 inches^4, and the radius of gyration of the section is $r=(33616/54.72)^{1/2}$= 7.82 inches.

Lastly by the column formula of the specifications. P- 8500-45x469.8/7.82= 5790. The maximum equilvalent live load stress in the end post is 313000 pounds, and the required sectional area is 313000/5790= 54.06 square inches. The above mentioned section will therefore be adopted.

The wind stresses are not large enough to affect the area required to resist fleture in the plane of the truss, but they will be considered in computing the stresses due to transverse fleture.

The end posts form a part of the portal which resists the wind pressure carried by the upper lateral system to the portal strut. The end posts bend in the plane containing their center lines.

In this case the inclined distance from the lower pin

of the end post to the bottom of the portal strut is assumed to be 35 feet or 420 inches. The point of inflection is at the middle of this length. The horizontal forces applied at the reactions of the portal are found to be 3750 pounds, according to the above computations. The moment due this force is $M= 3750x420/2=78750$ inch-pounds. Referred to theabove axis the section can stand a live loadmoment of $M=FI/Y=\frac{10000x3361.6}{8.76}$ $=3830000$ inch-pounds.

Since the moment due to the wind is less than 30 percent of the allowable live load bending moment, it need not be therefore considered.

CENTER LINE OF PINS.

The pins will be placed at such a distance below the center of gravity that the direct stress acting along the neutral axis will produce a moment neutralizing the moment due to the weight of the member itself. Let this distance be denoted by p, let W be the total weight of the member in pounds, l the length in inches, and P the total stress in the member, which in this case is the sum of the dead and live load stresses. Then $Pp= Wl/8$, or $p= 1/8\ Wl/p$.

Let d be the distance of center line of pins above the center of web plates. Then $d= l-p$.

To determine the weight per linear foot of a member, the weight of material in the section is taken and 20 percent added for the weight of Catten plates, lattice bars, rivet heads, and pin plates. For example, for the end post $U_1 L_1$ the weight per linear foot is,

2 web plates, 18"x3/16"- 68.88 pounds ,

1 cover plate, 26"x7/16"- 38.68 "

4 angles, 3 1/2"x7/16"- 39.20 "

2 flats 5x1- 34.00 "

and the sum of these plus 20 percent is 226 pounds nearly. Here the component which causes bending is 226/1.302 or 174 pounds, and the total weight W is 174x39.06= 6786 pounds. The distance p is p- 12x6786x39.06/8x31300= 1.26 inches, and hence d= 0.68-1.26= 0.58 inch is the correct distance of the center line of pins below the center line of web plates. In like manner are found p= 0.4 and d= 0.44 inches for $U_1 U_2$

Design of Pins.

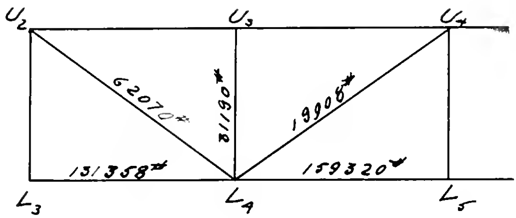
Where there are a number of bars on one pin and the forces are acting in different directions, it is necessary to resolve these forces into forces in two planes at right angles to each other and after finding the bending moment in each of these planes the resultant of these two moments at any point is to be taken as the total moment on the pin at that point. The resultant being equal to the square moments in the two planes.

Following are the methods for figuring the moments on the pins.

To determine the maximum bending moment on the pin at the joint L_4 the moment is figured with a maximum stress on the diagonal $U_2 L_4$ also with a maximum stress on chord $L_3 L_4$. The bending moment is figured for each of the above conditions.

Maximum stress in $U_2 L_4$ occurs when wheel 3 is at L_4. With

this loading the stresses in members on pin L_4 are shown in sketch and moments are given below.



Thus the horizontal bending moments are found as follows:

Member	Stress	V	X	VX	M
$L_3 L_4$	65679	—	—	—	—
$L_4 L_5$	79660	65679	1.125	73584	73584
$L_3 L_4$	65679	13981	1.125	15658	57926
$L_4 L_5$	79660	51698	1.125	57904	115830
$U_4 L_4$	12800	27962	1.125	31360	84470
$U_2 L_4$	40762	40762	.82	32616	51854
$U_3 L_4$	0	0			

while the vertical bending moments are:

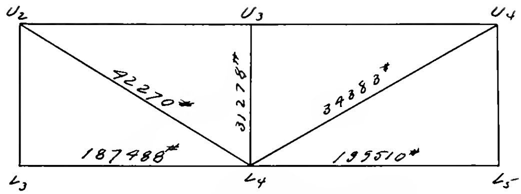
Member	Stress	V	X	VX	M
$U_4 L_4$	14800				
$U_2 L_4$	31190	14800	1.125	16650	16650
$U_3 L_4$	45990	45990	.82	36800	53450

The resultant bending moment is $M = (\overline{115830}^2 + \overline{53450}^2)^{1/2} = 127500$ pounds-inches, and referring to tables of manufacturers hand back it is found that a 4 1/2 inch pin resists a bending moment of 134200 pound-inches.

In order to provide adequate having area for the eye-bars, the pin cannot be less than 210800/2x12500x 1 3/8= 6.3 inches for 8- inch eye-bars. Therefore a 6 1/2 inch pin will be required at this point.

The maximum stress in the chord L_3 L_4 occurs when wheel 7 is at L_4.

With this loading the stresses or members connecting on L_4 are as shown below.



Horizontal Moments.

Member	Stress	V	X	VX	M
L_3 L_4	93744				
L_4 L_5	99755	93744	1.125	117180	117180
L_3 L_4	93744	6011	1.125	6734	110446
L_4 L_5	99755	87733	1.125	114052	3606
L_4 U_4	22000	12022	1.125	156286	159892
U_2 L_4	34022	34080	.82	27270	187162
U_3 L_4	0	0			

Vertical Moments.

Member	Stress	V	X	VX	M
U_4 L_4	5838				
U_2 L_4	25440	5838	1.125	7590	7590
U_3 L_4	31278	31278	.82	25648	33238

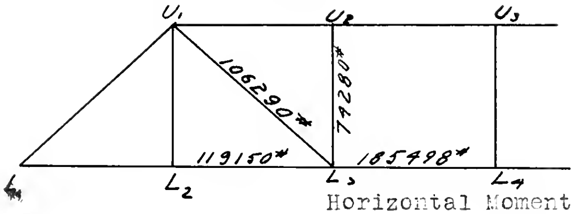
The resultant moment is $M= (187162^2+33238^2)1/2$=190000 poubd-

inches. A pin of 5 1/8 inch in diameter will resist a bending moment of 198200 pound-inches, and therefore it can be used. Since the bearing at this point requires a pin of 6.4 inches in diameter, a 6 1/2 pin will therefore be adopted.

Design of pin at the point L_3.

In a similar manner the bending moments on this pin is figured with the maximum stress in L_2L_3 and also with the maximum stress in U_1L_3

The maximum stress in L_2L_3 occurs when wheel 4 is at L_3. With this loading the stresses in the members are shown in sketch and moments are given below.



Horizontal Moments.

Member	Stress	V	X	VX	M
L_3L_4	92749				
L_2L_3	119150	92749	1.25	115940	115940
L_3L_4	92749	26401	2.75	72.660	43280
U_1L_3	66348	66348	1.20	79618	122898
U_2L_3	0	0			

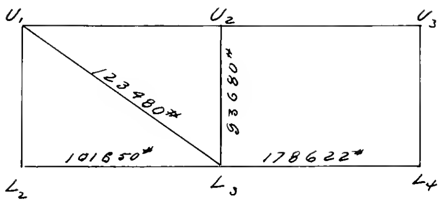
Vertical Moments.

Member	Stress	V	X	VX	M
U_1L_3	7428				
U_2L_3	7428	74280	1	74280	74280

and the resultant moment is $M = (\overline{122898}^2 + \overline{74280}^2)1/2 = 142300$ pound-inches.

The maximum stress in diagonal $U_1 L_3$ occurs when wheel 4 is at L_3.

The stresses due to this loading are shown in the sketch and the moments are represented in tabular form as follows:



Horizontal Moments.

Member	Stress	V	X	VX	M
$L_3 L_4$	89311				
$L_2 L_3$	101650	89311	1.125	111625	111625
$L_3 L_4$	89311	12339	2.75	339350	227725
$U_1 L_3$	76972	76972	1.20	92280	135445
$U_2 L_2$	0	0			

Vertical Moments.

Member	Stress	V	X	VX	M
$U_1 L_3$	93680				
$U_2 L_3$	93680	93680	1	93680	93680

The resultant moment is M= (227725-93680) 1/2= 245400 pound-inches. It is therefore found that with the maximum stress in the diagonal $U_1 L_3$ the pin will have a larger bending moment. A 5 5/8 inch pin will resist a bending moment of 262100 pound-inches and therefore may be used.

But the bearing on 8 inch eye-bar requires a pin of 6.4 inches, therefore a 6 1/2 pin will in this case also be used.

Since the bending moments on the other pins are less than those of the above mentioned, a uniform size of pins will be adopted and therefore a 6 1/2 inch pin for all joints will be used.

Lateral Bracing.

Since the stresses in the top lateral are very small, they will not be design, but mimimum allowable angles will be used. With regard to the bottom laterals they will be designed in the following manner.

The live stress in the diagonal $L_1' L_2$ is 49760 pounds, while the dead load stress is 16800 pounds. The net area for the live load stress is 49760/12000= 4.15 square inches. The net area required for the dead load is 16800/18000= 6.93 square inches. The total net area required is 5.08 inches. Let 2 angles, 3 1/2" x 3 1/2" x 1/2" be tried. Deducting two rivet holes for each angle thenet area obtained is 2x3.25-1.00= 5.5 square inches and therefore they will be used. The value of rivets in single shear per square inch is 1.5x9000=13500 pounds. The value of 7/8 inch rivet in single shear is .601x13500= 8100 pounds. The number of rivets required is therefore 66500/8100= 9. Since the rivets are field ones, their number will be therefore 1.5x9= 14.

In a similar manner it was found that 2 angles, 3 1/2" x 3 1/2" x 5/16" ought to be adopted for the diagonal $L_2' L_3$ and the number of field rivets required to connect them to the plate must not be less than 20.

In the same way it was found that only 1 angle is re-

quired for the diagonal L_3L_4' and the number of field rivets required is 8.

Design of Portal Bracing.

Since the horizontal angles in the upper and lower flange of the portal will consist of the same size, the maximum tension or compression of the member will be used. The largest compressive stress occurs in member DQ. The length of the member is 72 inches. The allowable unit compressive stress is P= 13000-60l/r= 13000-60x72/r. Let angles, 3 1/2"x 3 1/2"x5/16" be tried. The radius of gyration of this angle is 1.08 and the unit compressive stress is therefore P= 13000-60x72/1.08= 9000 pounds. The area required is 26930/9000= 2.99 square inches. The area of two angles is 2x2.09= 4.18 square inches, which gives a excessive area. But these angles are the minimum allowable and therefore they will be used. The value of 7/8" rivet in single shear at 13500 pounds per square inch is 8100 pounds. The number of rivets required for connection of the angles to the plate is 26900/8100= 4. In the same manner it was found that the angles of the lower flange of the portal will be of the minimum allowable ones and therefore the same as above will be used. The tension in the middle diagonal is 12040 pounds, while the compressive stress in the other interested diagonal is also 12040 pounds. Let 1 angle, 3 1/2"x 3 1/2"x5/16" be tried. The required area for tension is 12040/12000= 1.003 square inches, while the area required for compression is 12040/9000= 1.34 square inches. Therefore the above mentioned angle will be used.

In the same manner it was found that a minimum allowable angle will satisfy the conditions of the other members and therefore it will be used.

Design of Pin Plates.

The maximum pin bearing at the bottom of the post $U_2 L_3$ equals the maximum vertical component in the diagonal $U_1 L_3$ which is 210800 Sin 50°10′=156000 pounds. As the diameter of the pin is 6 1/2 inches, the bearing area required on each side of the post is 156000/2x6.5x12500= 6.975 inches. The thickness of the channel web is 0.636 inches, and hence one pin plate whose thickness is 3/8″ will be required.

The full bearing value taken by the pin plate is 0.375x 6.5x12500= 30000 pounds.

The shearing value of a 7/8″ rivet in single shear at 9000 pounds per square inch is 5400 pounds. The number of rivets required is then 30000/5400= 6. Additional rivets are placed below the pin to keep the parts in contact.

At the upper panel point the maximum bearing value on the pin is the stress in the post $U_2 L_3$, which is 87118 pounds. The bearing area required is 87118/2x6.5x12500= 0.544 inches. Since the thickness of the channel web is 0.636 inch, no pin plate is therefore needed.

Since the suspender $U_1 L_2$ is a tension member, its net sectional area at the pin hole must be 40 percent in excess of the net area in its main body. The area for each side is therefore 14.92x1.4/2= 10.44 square inches. The simplest arrangement is to use one pin of 10.5 square inches. The net

area of the pin platr is 10.5-6.5x.875= 4.8 square inches, and its full tensile stree is 10000x4.8= 48000 pounds. The shearing value of 7/8" rivet at 9000 pounds per square inch is 5400 poumds. The number of rivet required is 48000/5400 = 9 =G. The distance beyond the pin is $\frac{10.5x.70}{.875-.636}$=4.9 inches.

The nêt area at the pin holes in the lower chord $L_2 L_3$ must not be less than 20.39x1.4/2= 14.28 square inches.

One pin plate, 17"x7/8", will be used, whose gross area is 14.8 square inches. The net area of the pin plate is 14.8-6.5 x.875= 9.19 square inches. The full tensile stress of the pin plate is 9,19x10000= 91900 pounds. The shearing value of a7/8" rivet in single shear is 5400 pounds.

The number of rivets required is 91900/5400=18, and the distance beyond the pin is therefore $\frac{.70x14.8}{.875-.5}$= 7 5/8 inches.

The pin bearing at panel point U_2 in the upper chord is to be desigred to take the horizontal component of the stress in the diagonal $U_2 L_4$ or 71190 pounds. The linear bearing on each side is 71190/2x6.5x12500= 0.44 inches and hence a pin plate of minimum thickness will be used. The share of stress taken by the pin plate is 6/13x71190/2= 16400 pounds and the number of rivets required is 16400/5400= 4.

The pin bearing at panel point U_1, in the upper chord takes the stress of the chord $U_1 U_2$, which is 318725 pounds. The linear bearing an each side is 318725/2x6.5x12500=1.991 inches. The thickness of the web plate is 437 inch, and thickness of pin plates must not be less than 1.991-.437=1.55 inches.

Two pin plates, each of 13/16" will be selected. The

stress taken by the pin plates is 1.624x6.5x12500= 128000 pounds. Since the rivets are in double shear, the bearing value of rivet will be considered. The bearing value of 7/8" inch rivet in a 7/16 inch plate at 15000 pounds per square inch is 5800 pounds. The number of rivets is therefore 128000/5800= 23.

Finally, the pin bearing at panel point U_1, in the end post takes the equivalent live load stress of the end post which in this case is 313000 pounds. The linear bearing on each side is 313000/2x6.5x2500= 1.96 inches.

The thickness of the pin plates must not be less than 1.96-.562= 1.398 inches. Two pin plate, each of 3/4" in thickness will be selected. The full bearing stress of the pins is 1.5x6.5x12500= 120000 pounds. The bearing value of 7/8 inch rivet in a 9/16 inch plate is 7350 pounds. The number of rivets is therefore 120000/7350= 17.

End Bearings.

The maximum reaction is equal to 3 times the dead panel load divided by 2, plus the maximum live load reaction when the bridge is fully loaded. The maximum reaction is therefor e 3x33750/2+226450= 282075 pounds.

The design of the pedestals for the fixed end will be made first. The bearing area required is 282075/12500= 22.56 square inches, and 1/6.5x22.56= 3.5 inches is the width of the bearing area on a 6 1/2 inch pin. Four vertical bearing plates each 7/8 inch thick will be used. The inside connection angles will be 5"x6"x7/8, and the outer ones will be 6"x6"x7/8".

The allowable bearing value of massury per square inch was taken as 250 pounds. The bearing are required is 282075/250= 1128 square inches.

If the width of the massury plate be taken at 30 inches, the length therefore must not be less than 38 inches. The bearing plate will be the same area and thickness. Both bearing and masonry plates will be ordered 13/16 inch thick and finished on one side to 3/4 inch. The pedestal will be onchored to the masonry by 1 1/2 inch anchor bolts securely fat-bolted in the masonry to a depth of 12 inches.

The design of the pedestal and roller next for the free end is as follows: The vertical plates and connections angles will be the same as the fixed end. The width cannot be less than 22.5 inches. The allowable load for rollers per square inch is 300d. Here d is 4 7/8+0.5= 5.375 inches, which makes the load per linear inch equal to 1612.5 pounds. Hence, 282075/1612.5= 174.9 linear inches are required. If each roller be 30 inches long, six will be needed. The bearing and masonry plates will be ordered 13/16 inches thick and finished on one face to 3/4 inch. The dimensions of the bearing plate will be the same as the masonry plate.

Analysis of Weight.

Intermediate posts:

2- U_1U_2-	2x2x31x35-	4340	
2- U_2L_3-	2x2x31x35-	4340	
1- U_3L_4-	1x2x31x20.5-	2540	11220 pounds

Diagonals:

2- U_1L_2-	2x2x39.5x37.4-	5900	
2- U_2L_4-	2x2x39.5x20.4-	3220	
2- U_3L_3-	2x2x39.5x10.2-	1610	10730 pounds.

Lower chords:

4- $L_1 L_2$ - 4x2x26.5x57.2-	12120	
2- $L_3 L_4$ - 2x4x26.5x272.2-	5770	17890 pounds

Upper chords:

2- $U_1 U_2$ - 2x2x26x69.95-	7280	
2- $U_2 U_3$ - 2x2x26x72.88-	7580	14860 pounds

End posts:

2- $U_1 L_1$ - 2x2x39.5x81.86-		12940 pounds

Pins:

12- 6 1/2 inch pins-12x2x112.8-		2680 pounds

Top lateral bracing.

6 angles, 3 1/2x3 1/2x3/8x25.5x8.5-	216.75	
4 angles, 3 1/2x3 1/2x3/8x15.5x8.5-	131.75	348.5 pounds

Bottom lateral bracing:

4 angles, 3 1/2x3 1/2x1/2x25.5x8.5-	216.75	
2 angles, 3 1/2x3 1/2x5/16x25.5x12.4	316.25	523 pounds

Portal bracing.

2 angles, 3 1/2x3 1/2x5/16x7.5x8.5-	63.75	
2 angles, 3 1/2x3 1/2x5/16x8x8.5-	70.	
2 angles, 3 1/2x3 1/2x5/10x6x12.4-	75.	
2 angles, 3 1/2x3 1/2x5/16x8.5x8.5-	64.25	
1 angle, 3 1/2x3 1/2x5/16x8x8.5-	70.	283 pounds

Sway bracing.

2 angles, 3 1/2x3 1/2x3/8x7.75x8.5-	69.5	
2 angles, 3 1/2x3 1/3x3/8x7x8.5-	59.5	
3 angles, 3 1/2x3 1/2x3/8x8.5x8.5-	64.	193 pounds

End bearings.

1 Plate, 30 1/2x1 1/8x2-8x114.85-	342
1 plate, 30 1/2x 1 1/4x3-8x127.5-	508
2 angles, 6x6x7/8x2-6x31-	77
2 angles, 6x6x7/8x2-6x31-	77
1 plate, 10 1/4x3/8x1x13-	13
2 plates, 19 1/2x1/2x2-6x32-	80
6 Plates, 17x5/8x2-6x36-	90

Since there are 2 shoe bearings on each truss the amount is 2x1187- 2374 pounds

Minor details:

Pin plates	1500	
Tie plates and lacing	2500	
Connections, splices and etc.	2400	6400 pounds

Floor system:

6 stringers at 5000	30000	
3 1/2 floor beams at 4600	16100	46100 pounds

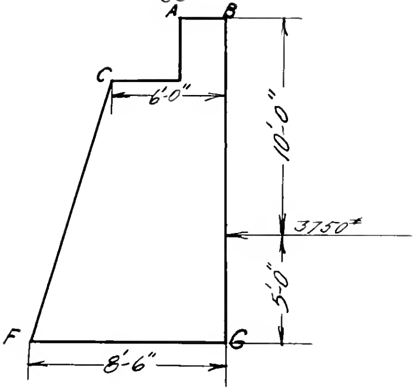
The total weight of one truss is 126550 "

The total weight of the superstructure is therefore

8x126550..................................1012400 pounds.

4. DESIGN OF PIERS AND ABUTMENTS.

Since the design of one of the piers was briefly outlined above under the head of the selection of design, it will not be farther regarded. Owing to the lack of actual data the design of all the other piers will be the same. The design of one of the abutments will be considered next. The height of the abutment will be assumed to be 15 feet. A sketch of it is given below and the abutment is computed in the following manner.



The filling behind the wall consist of sand. The weight of a cubic foot of sand was assumed to be 100 pounds, and the angle of repose of the material to be 30 degrees. If p be the unit intensity of the vertical pressure of the earth and g the horizontal thrust, then for equilibrium the formula will be $\frac{p}{q} = \frac{1-\sin\varphi}{1+\sin\varphi} = 1/3$

Considering the section of the wall to be one foot in length the average horizontal pressure is $q = 100x5/3x2 = 1500/6$ pounds and the total pressure applied at 1/3 of theheight from the base is found to be 1500x15/6= 3750 pounds. By taking moments about the base the overturning moment due to this pressure is 3750x5= 18750 foot-lbs or 225000 inch-pounds. The top and bottom dimensions of the section are also given in the sketch. The centre of gravity of section will be found next, by taking moments about BG and the equation becomes 70x= 5x5+ 45x2.25+205(4.5-4/3), and hence X= 3.15 feet, i.e. the distance of the center of gravity of the section from the point G is 3.15 feet.

The volume of the wall for 1 foot of length was found to be

117.6 cubic feet and assuming the weight of one cubic foot of concrete ~~wall~~ to be 150 pounds, the weight of the wall is 150x117.6- 17640 pounds.

The tangent of angle that the resultant of the horizontal and vertical pressure makes with thevertical is 3750/17640= 0.21; and the angle is 11°50', which shows that it is safe enough with the middle third. Sliding cannot place if the weight of the wall times the coefficient of friction is equal or more than the horizontal pressure. In this case it is safe enough for sliding.

In order to test the abutment for overturning the center of moments will be taken about the outer edge. The overturning moment due to the horizontal pressure is 3750x5= 18750 foot-lbs., while the resisting moment of the wall is 17640x5= 88200 foot-lbs.

The wall is therefore safe enough for overturning. The determination of the bearing area will be considered next. In this case it is necessary to take into account the live load reaction. The portion of the wall resisting this force is 11.25 feet. The reaction can be considered as uniformly distributed over that portion. The total vertical force acting on the foundation is 17640+282000/11.25= 44260 pounds.

. The bearing value of sand per square foot was taken as 6000 pounds. Therefore the required bearing area is 44260/6000 = 7.38 square feet. The area of the wall at thefoundation is 8.5x1- 8.5 square feet, and therefore no piling for the abutment is required.

5. COST.

The cost of one pound of metal is taken as 4 cents, and the total cost of superstructure is therefore 0.04x1012400= $40496. By looking through the cost of different railroad bridges it was found, that an average value of $2.00 can be taken for a ton of metal to be painted. Therefore the cost of painting is 2x1012400/2000= $1012. The cost of erecting pin connected bridges is also a nearly constant quantity and it varies from 0.7-1.2 cents per pound. A value of 0.75 cents per pound will be selected in this design.

The cost of erection is therefore

0.0075x1012400= $7593.

The volume of concrete of one abutment including two wing walls and parapet wall was found to be 222.2 cubic yards, and assuming the cost of a cubic yard of concrete and its laying to be $6.80, the total cost of one abutment is 6.80x222.2= $1510. The cost of tw land abutments is therefore 2x1510= $3020. The cost of three piers was found to be $10934. The total cost of substructure is therefore $13954.

Cost of Substructure..........................	$13954
" " Superstructure........................	40496
" " Erection.............................	7593
" " Painting.............................	1012
Total cost of structure......................	$63055

SPECIFICATIONS FOR SUBSTRUCTURE.

General Description

1. The work to be done under these specifications comprises the building of three piers and two abutments.

2. All the piers shall be founded on solid rock. The elevations given in the plans are approximate only. The Engineer may, as the work proceeds, require the foundations to be placed either at higher or at lower elevations than named herein.

Materials.

3. Timbers for caisons or cofferdams shall be either long or short leaf pine, sawed accurately, free from rot, splits, shakes or other imperfections which in the opinion of the engineer may impair its strength or durability.

4. Steel for rods and drift bolts shall be of soft steel and shall be subject to the specifications of soft steel for superstructure.

5. The cement will be furnished by the Bridge Company, but the contractor will be held responsible for all waste after it is delivered to him from the Company's warehouse.

6. Sand for concrete shall be clean, sharp, coarse river sand, or other sand of equal quality in the judgment of the Engineer.

7. Broken stone shall be of hard sound, clean limestone. It shall be broken by machine and screened in a rotary screen which shall remove all dust and fragments which will pass

through holes three-eighth inch in diameter and prices exceding one and one-half inches in diameter.

8. In proportioning material for conrete one volume of cement shall be taken to mean 380 pounds net, one volume of sand or broken stone shall be taken to mean 3 1/2 cubic feet packed or shaken down.

9. Measurements of sand and broken stone shall be made in barrels or boxes.

10. In preparing mortar the specified amounts of cement and sand shall first be mixed dry to a uniform color. The water shall then be added in such a manner as not to cause any washing of the cement, and the mixing proceeded with until the mortar is thoroughly mixed and uniform in appearance.

11. Wherever possible concrete shall be mixed with a machine approved by the Engineer. Preference will begiven to a machine which will mix concrete in catches, the cement, sand and broken atone, measured as specified above.

12. Concrete shall be deposited in the work in such a manner as not to cause the partial separation of the mortar and stones. It shall be spread in horizontal layers from six to twelve inches in thickness and thoroughly rammed. The rammers shall weigh at least twenty pounds.

13. The consistency of the concrete shall be as required by the Engineer from time to time, but will generally be such that the concrete will quake under hard ramming.

14. No mortar or concrete shall be used after it has

begun to set; when setting commences the material inquired shall be immediately wasted.

15. All concrete in the piers shall be in the proportion of one vokume of cement to two and one-hald volumes of sand and six volumes of broken stone.

16. The concrete in the copings shall be in proportion of one volume of cement to two volumes of sand and four volumes of broken stone.

17. A facing of mortar shall be put in next to the molds of all concrete work for all piers and abutments.

SPECIFICATION FOR PORTLAND CEMENT.

1. The cement used in the substructure will be Portland Cement, manufactured at works which have been in successful operation for at least two years.

2. The cement shall be manufactured from a mixture of calcarous and clayey earths or rocks and shall contain no furnace slag, gray limestone, hydralic lime or trass.

3. The average weight of a barrel of cement shall be at least 380 pounds, net.

4. Samples of cement for testing will be taken from the interior of the packages in such manner and in such number as the Engineer may direct. The test will be made on the individual samples without intermixing.

5. The cement shall not contain more than two percent of sulphuric acid or more than three per cent of magnesia.

6. The cement shall be so finely ground that at least 97 percent by weight will pass through a standard sieve having ten thousand openings per square inch.

7. The time required for setting will be determined with mortars in which the weigh of the water shall be 20 percent of the weight of the cement mixed to a plastic condition, formed in suitable moulds and kept at a temperature of from 65 to 70 F. Mortar will be considered to have taken its initial set when it will sustain a wire 1/12 inch in diameter loaded to 1/4 pound without breaking the surface of the mortar; it will be considered to have taken its final set when it will

sustain a wire 1/24 inch in diameter loaded to one pound without breaking the surface of the mortar. The initial set shall not be taken in less than thirty minutes; the final set shall be taken in eight hours or less.

8. The test of constancy of volume shall be made on a similar mortar formed on glass into a cake about 3 inches in diameter and 1/2 inch thick at the center, worked doen to a think edge all around. It shall be subjected to one of the following tests:

(a) the cake shall be left in air until it takes the final set and shall then be placed in water maintained at a temperature of 60 to 80 F. for a period of 28 days, or

(b) the cake as soon as formed shall be placed on a rock in the upper part of a covered vessel partly filled with water, which shall be maintained at the temperature of 110 to 115 F., so that the mortar will be in warm, moist air while setting. After having been thus exposed for 6 hours the cake shall be immersed being maintained. The test (a) shall be applied when sufficient time is available for its completion. If sufficient time for test (a) is not available, test (b) shall be used.

9. The test for tensile strength will be made on a mortar containing one part of cement to three parts of standard crushed quartz sand by weight. The quarz shall be of such fineness that all of it will pass through a standard sieve having 400 openings per square inch. Enough water shall be used to form a stiff mortar. The mortar shall be formed into a briquette having a minimum section at the center of one

square inch. It shall be left under a damp cloth for 24 hours and then immersed in water maintained at a temperature of 60 to 80 F. At the age of 28 days it shall be removed from the water and immediately broken by tensile strain. If the average strength of the brignettes from any shipment is less than 240 pounds the cement will be rejected. If any number of brignettes less than 1/5 break at 200 pounds or less the packages from which these brignettes were made will be again tested, and if any brignettes fail to sustain a tensile strain of 200 pounds the entire lot will be rejected.

10. The tests above specified will be made by the agents of the Bridge Company, under the direction of the Engineer.

11. Rejected cement shall be removed from the warehouse by the Contractor within five days or receipt from the Engineer of notification of rejection, and at the Contractor's sole expense.

SPECIFICATIONS FOR SUPERSTRUCTURE.

1. General Description.

1. The superstructure is divided into four equal spans. The distance between centers of end pins of each span is 150 feet. The depth of these spans is also equal and is 30 feet between the centers of pins.

2. The trusses will be spaced 17 feet apart between centers. Each span is divided into six panels. The length of each panel is 25 feet.

3. The wooden floors will consist of transverse ties or floor timbers. They shall be spaced with openings not exceeding six inches, and shall be notched down 1/2 inch and be occured to the supporting girders by 3/4 inch bolts at distances not over six feet apart. There shall be guard timbers on each side of each track and notched one inch over every floor timber with a half-and-hald joint of six inches lap. Each guard timber shall be fastened to every third floor timber and at each splice with a 3/4 inch bolt. The guard and floor timbers must be continuous and properly supported over all piers and abutments.

4. The estimated approximate weight of the superstructure is 507 tons.

Plans.

5. Full detail plans showing all dimensions will be furnished by the engineer. The work shall be built in all respects accordingly to the plans. The contractor, however,

will be expected to verify the correctness of the plans and will be required to make any changes in the work which are necessitated by errors in the plans, without extra charge, where such errors could be discovered by an inspection of the plans.

II. Material.

6. All parts, except nuts, swivels, elevises and wall pedestal plates, will be of steel. The nuts, swivels and elevises may be of wrought iron. The pedestal plates will be of cast iron.

7. All material shall be subject to inspection at all times during its manufactur, and the engineer and his inspectors shall be allowed free access to any work in which any portion of the material is made. Timely notice shall be given to the engineer so that inspectors may be on hand.

Steel.

8. Steel will be divided into three classes: first, High Grade Steel, which shall be used in all the principal truss members; second, Medium Steel, which shall be used in the floor system, laterals, portals, transverse bracing and the lacing of the truss members; third, Soft Steel, which shall be used only for rivets and at the option of the contractor where wrought iron is permitted.

9. All steel must be uniform in character for each specified kind.

The finished bars, plates and shapes must be free from

injurious seams, orflows, cracks on the faces or corners.

11. The tensile strength, elastic limit and ductility shall be determined by samples cut from the finished material after rolling. The samples to be at least 12 inches long. and to have a uniform sectional area not less than 1/2 square inch.

12. The elongation shall be measured on an original length of 8 inches. At least two test pieces shall be taken from each melt or blow of finished material, one for tension and one for bending.

13. A piece of each sample bar shall be bent 180 degree and closed up against itself without showing any crack or flow on the outside of the bent portion.

14. Every melt which does not conform with the requirements shall be rejected.

15. A full report of the labaratory test shall be furnished certified by an inspector accepted by the Chief Engineer.

16. The broken and bent specimens shall be preserved subject to the orders of the Chief Engineer.

17. Analysis shall be made by the manufacturer of every melt, showing amount of phosphorus, carbon, silicon and manganese.

18. The cross-section shall never differ more than two percent from the ordered cross-sections as shown by the dimensions on the plans.

19. All sheared edges shall be planed off so that no rough or sheared surface shall ever be left on the metal.

20. Steel for pins shall be sound and entirely free from piping. All pins in the main trusses shall be drilled through the axis.

III. Manufacture.

21. The work shall be done in all respectd according to the detail plans furnished by the chief Engineer.

22. All surfaces in contact shall be cleaned and painted before they are put together.

23. All work shall be finished in the shop and ample time given for inspection.

24. No material shall be loaded on cars until accepted by the inspector.

25. The finishing of work after loading will not be permitted.

Solid Drilled Work.

26. All riveted members which are made of High Grade Steel and all other pieces connecting with such members shall be solid drilled, no punching whatever being allowed, excepting lacing bars which may be punched and reamed.

27. The size of rivets shown on the plates is the size of the cold rivet before heating.

28. The diameter of the finished hole shall never be more than 1/16 of an inch greater than the diameter of the cold rivet. It is intended that the heated rivet shall not drop into a hole, but require a blow from a hammer to force it in it. If it found that the rivets will drop easily into the holes, the inspector will condemn those rivets and a larger size.

29. The riveted connections of the portals, cross-frames and floor beams with the post and chords shall be drilled with the several parts fitted together.

30. The field rivets shall be driven by power wherever this is possible.

31. All rivets shall be eegulated in shape, with hemi-spherical heads concentric with the axis and absolutely tight. Tightening by calking or recupping will not be allowed. This applies to both power driven andhand driven rivets.

32. All pin holes shall be drilled after all other work is completed.

33. All chord sections shall be stamped at each end on the outside with letters and numbers designating the joints in accordance with the diagram plan furnished by the Chief Engineer.

34. The same rule shall apply to the marking of the posts.

35. Pin holes in the posts shall be truly parallel with one another and shall be at right angles to the axis of the post.

36. Measurements shall be made from an iron standard of the same temperature as the member measured.

Forged Work.

37. The heads of eye bars shall be formed by upsetting and forging into shape by a process acceptable to the Chief Engineer. No welds will be allowed.

38. After the work is completed the bars shall be

annealed in a suitable annealing furnace by heating them to a uniform dark red heat and allowing them to cool slowly.

39. The form of the heads of the steel eye bars may be modified by the contractor to suit the process in use at their works, but the thichkness of the head shall not be more than 1/16 inch greater than that of the body of the bar, and the heads shall be of sufficient strength to break the body of the bar.

40. Nuts, swivels and clevises, if made of steel, shall be forged without welds, whether made of steel or wrought iron, one of each size shall be tested and be of sufficient strength to break the bars to which they are attached.

41. Eye bars shall be bored truly and at exact distances, the pin holes to be exactly on the axis of the bar, and at exactly right angles to the plane of the flat surfaces.

42. Pin holes shall be bared with a sharp tool that will make a clean smooth cut. Two cuts shall always be taken, the finishing cut never to be more than 1/8 inch. Roughness in pin holes will be sufficient reason for rejecting bars.

Machine Work.

43. All bearing surfaces shall be faced truly.

44. Chord sections shall be faced after they are riveted up complete. The end of the stringers and of floor beams shall be squared in a facer.

45. All surfaces so designated on the plans shall be plane.

46. All sheared edges shall be planed off, and all punched holes shall be drilled or reamed out.

47. The plabs show the distances between centers of pin holes. Shop measurements, however, shall be made between the bearing edges of the pin holes, that is, between the inside edge of compression members and the outside of tension members, with a proper allowance for the diameter of the pin.

48. The rollers shall have the hollow sides planed and the bearing surfaces turned to a perfectly true cylinder and polished.

49. The rods passing through the rollers shall fit the holes with a play not exceeding 1/32 of an inch.

Miscellaneous.

50. All material shall be cleaned, and, if necessary, scraped and given one heavy coat of Cleveland iron- clad paint, purple brand, put on with boiled linseed oil, before shipment. This applies to everything except machine finished surfaces.

51. The same paint shall be used wherever painting is required.

52. All small bolts, all pins, the expansion rollers and everything with special work on it, shall be carefully boxed before shipment.

53. The contractor will be required to furnish the field rivets for erection, furnishing 20 percent in excess of each size over and above the number actually required, but this excess will not be estimated, but considered as taking the place of the work which is not done on these rivets.

IV. Inspection.

54. The mill inspection shall be performed at the expense of the contractor, by an inspector accepted by the Chief Engineer.

55. This inspector will be required to furnish the certificates and notices in the manner specified above.

56. The mill inspector shall from time to time check the manufacturer's analyses by analyses made by an independent chemist.

57. The acceptance of material by such inspector will not be considered final, but the right is reserved to reject material which may prove defective or objectionable at any time before the completion of the contract.

58. The inspection at the shops will be under the charge of an inspector appointed by the Chief Engineer, with such assistance as may be required.

59. Such inspector will be considered at all timed the representative of the Chief Engineer, and his instructions shall be followed in the same manner as if given by the Chief Engineer.

Test of Eye- Bars.

60. In the case of bars too long for the machine, the bars shall be cut in two, each half reheaded, and both halves tested in the machine, the two tests, however, to count as a single test bar.

61. If the capacity of the machine is reached before the bar is broken, the bar shall be taken out of the machine

and the edges shall be planed off for a length of 10 feet at the center until the section is reduced to the equivalent of 16 square inches of section of the original bar. The bar shall then be placed in the machine andbroken; when this is done the elongation shall be measured on a length of 8 feet and an ultimate strength of, 6000 pounds computed on the 16 inches of the original section will be considered satisfactory.

62. The failure of a bar to break in the body shall ~~reet~~ not be considered sufficient reason for rejection, provided the required elongation is obtained and not more than one-third of the bars break in the head.

V. Terms.

63. The work will be paid by the pound of finished work loaded on card.

64. No material will be paid for which does not form a part of the finished superstructure.

65. All expenses of testing shall be borne by the contractor.

VI. Erection.

66. The contractor will be expected to receive all material as it arrives on the cars, to unload this material and store it in a material yard until ready for erection.

67. The contractor will be required to keep all the material in good condition, and in case of its becoming dirty or rusty, will be expected to clean it before erecting.

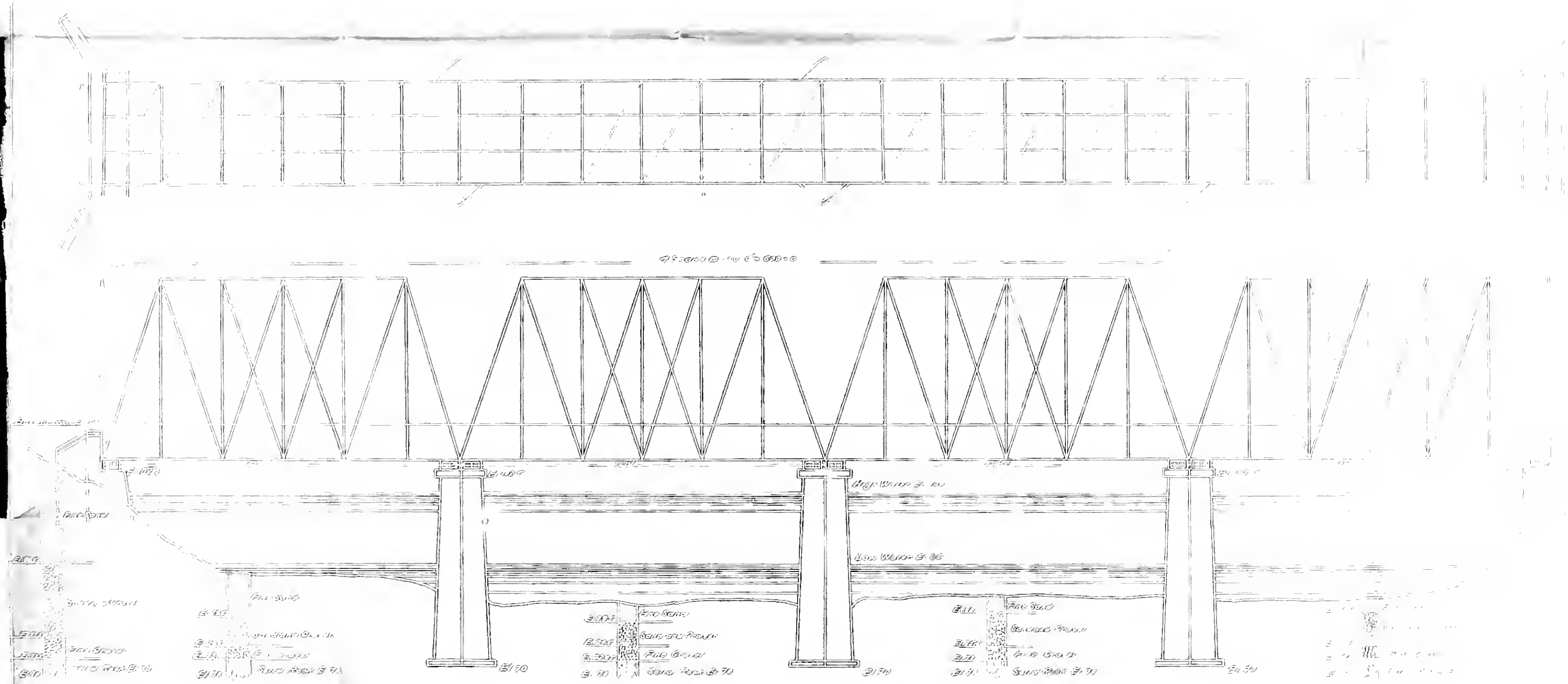
68. The contractor will be required to paint all surfaces which will be inaccessible for painting after erection, the paint being furnished by the Bridge Company.

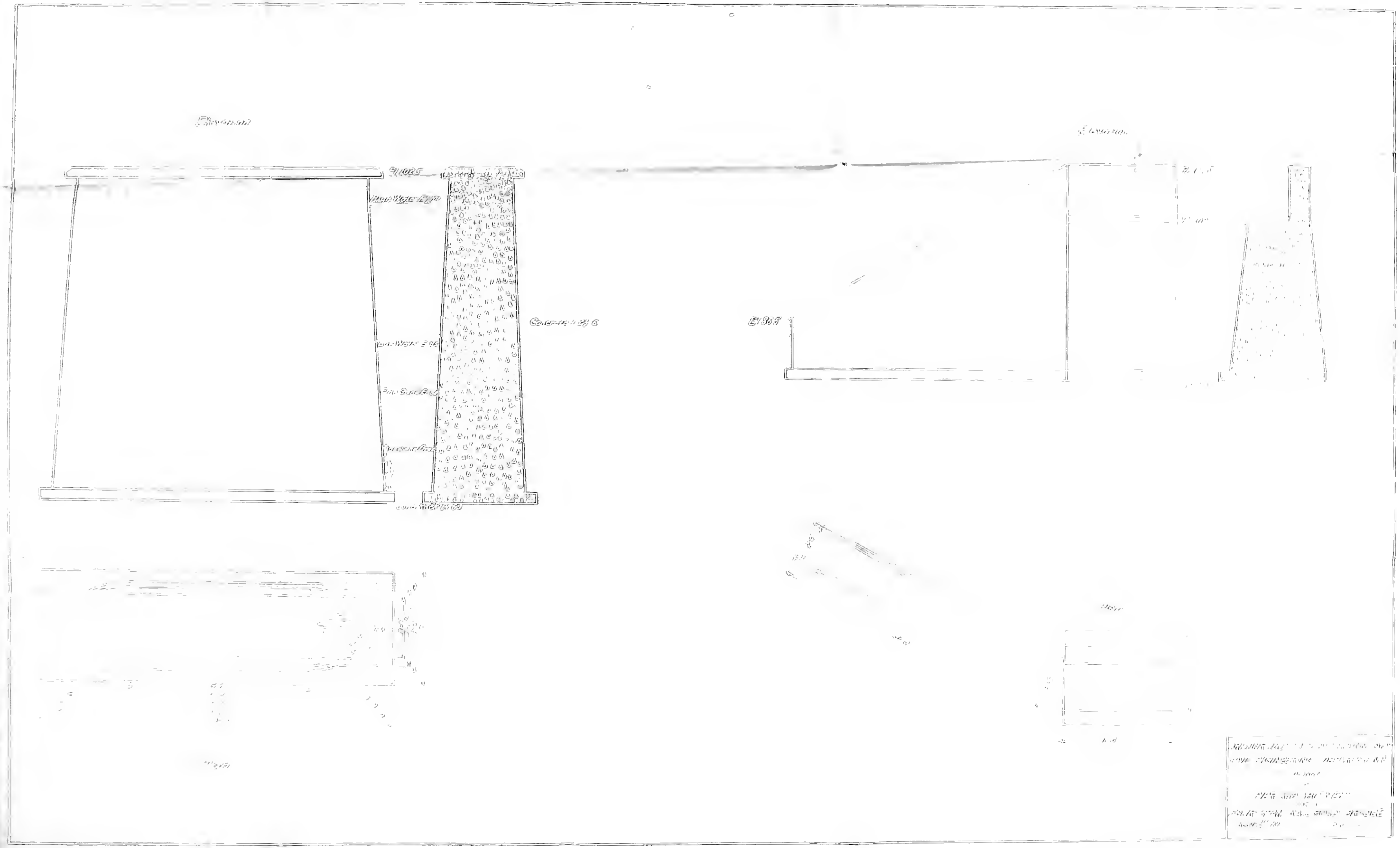
69. The contractor will be required to remoce all work which he may put in the river so that there will be nothing left either to interfere with navigation or to catch drift.

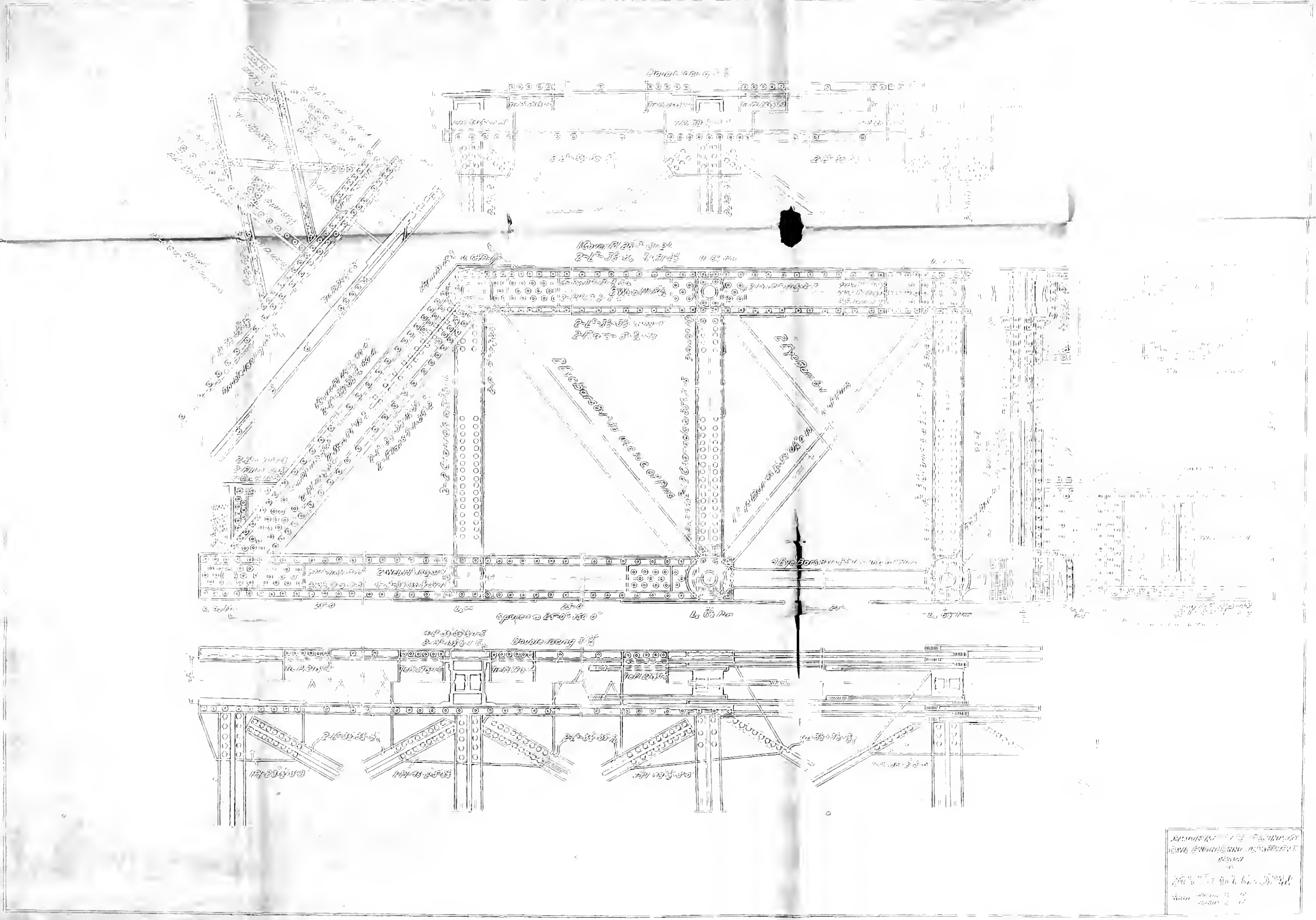
70. The contbractor will be required to erect the superstructure complete in every respect including riveting.

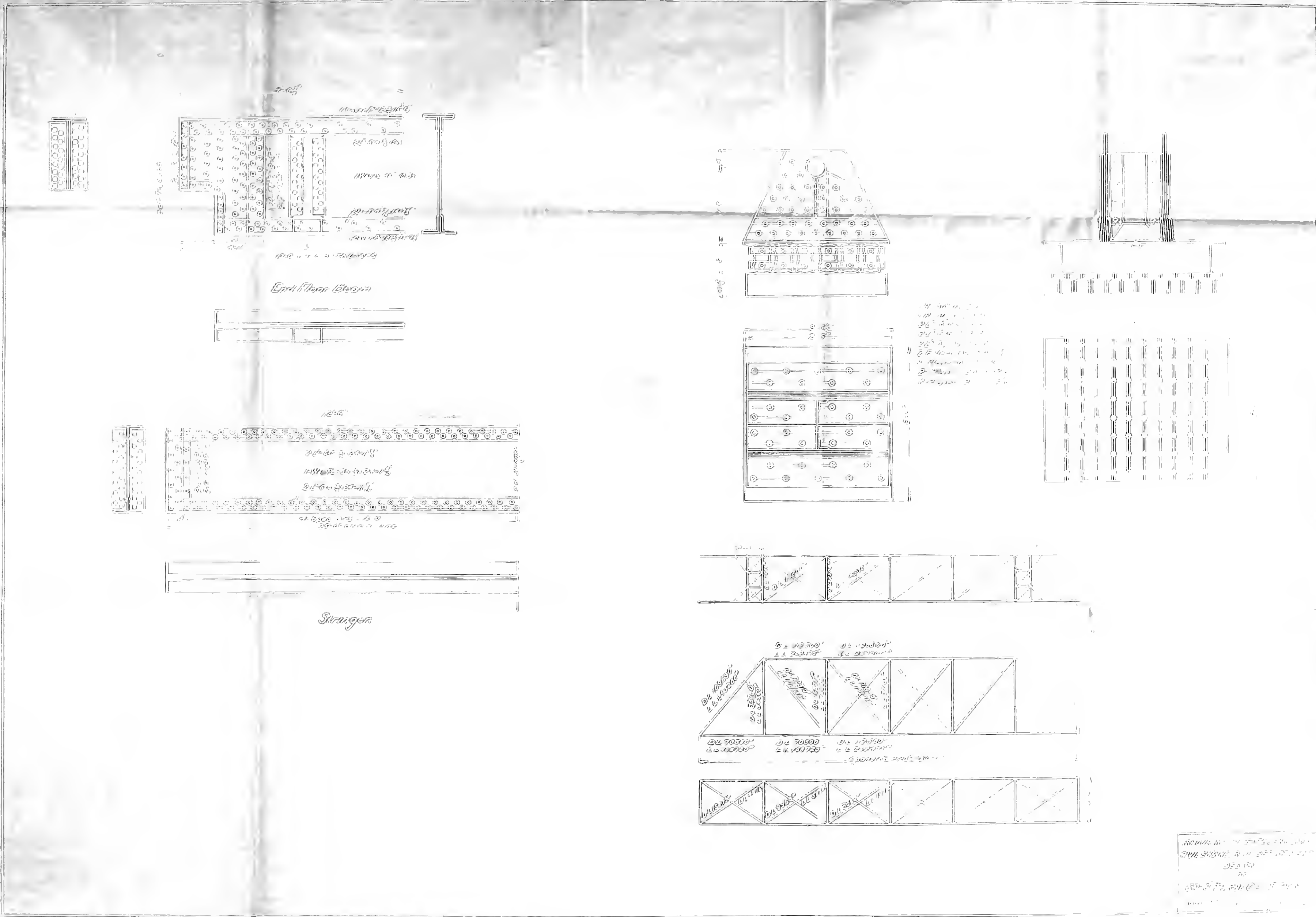
71. The expansion end of the span shall be adjusted so that the axis of the rollers will be exactly vertical at a temperature of 70 degrees F.. This adjustment shall be made at atime where there has been no sun on the steel work for ten comtinuous hours, and when there has been no sudden change of temperature.

GENERAL ELEVATION AND PLAN OF BRIDGE






Double lacing
2 Eye Bars
4 Eye Bars
Civil Engineering Department
Design


End Floor Beam
Stringer